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Paper 1047 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
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SOME DESIGN CONSIDERATIONS FOR OXIDATION PONDS

Earnest F. Gloyna,[1] A.M. ASCE, and Edward R. Hermann[2]
(Proc. Paper 1047)

SYNOPSIS

Oxidation ponds are economical. During the day the algae, acting as tiny aeration devices, substitute for air compressors, distributors, and diffusers all in one comprehensive system. Such waste stabilization systems should be particularly attractive to those smaller communities that use their effluent for irrigation. Design should be based upon proposed effluent usage, B.O.D. load, climatic characteristics and geographical location. Uniform inlet and flow-through distribution systems, bank stabilization and elimination of mosquito breeding places are of prime importance. B.O.D. loadings up to 200 Lb./Ac./D. are possible, but the actual value is dependent upon final disposition of the effluent. Concentrated and available nutriment in mineralized form can be controlled by dilution or by extending the algal-bacterial phase to include the predator-fish stage.

INTRODUCTION

It is the purpose of this paper to list some of the factors which may influence the operation of oxidation ponds and to relate some investigational results. Under arid, bright climates especially, the oxidation pond type of treatment produces an effluent that is well stabilized and free of odor. However, the design of these ponds involves a thorough understanding of sanitary engineering, aquatic biology and available energy budgets. The selection of appropriate structural facilities that are capable of creating a satisfactory environment is also necessary.

The oxidation ponds, better called waste stabilization or possibly Redox (Reduction-Oxidation) ponds, depend upon two primitive forms of life and the primary source of the earth's energy for their operation. The primitive life forms are the algae and the bacteria. The primary energy source is the sun.

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1. Associate Prof. of Civ. Eng., The Univ. of Texas, Austin, Tex.
2. Research Engr., San. Eng. Research Lab., The Univ. of Texas, Austin, Tex.

The energy from the sun together with the photosynthetic properties of the algae make it possible for the algae to use partially fermented waste, primarily carbon dioxide, a low energy compound, to produce more algal cells and free oxygen. The available dissolved oxygen and sewage thus stimulate the activities of the aerobic bacteria.

The use of algae presents a challenge to conservationists and engineers. In the United States the primary use of oxidation ponds is to treat and stabilize used-water, but these ponds also could be turned into protein factories. The oxidation pond might be operated to recover nitrogen and phosphorus that is ordinarily lost in conventional biological treatment processes. Renn estimates, on the basis of only 60 per cent operation efficiency, that a pond could produce 1,400 pounds of protein per day in the form of algal cells from the waste of 10,000 people.[3] Gotaas[4] reports that the possible annual yield of algae, primarily protein, might be thirty tons per acre per year. This quantity is roughly twenty to fifty times the rate for field crops.

The design of oxidation ponds must be evolved around the following three questions. First, is this a case in which the only criterion is that of treating putrescrible liquid without regard to effluent appearance? Second, is the question at hand one of waste stabilization and the production of a colorless effluent having a minimum amount of suspended solids? Third, is the design to be based upon waste treatment and maximum protein production? These processes, simple waste purification, algae control and maximum conversion of waste material to useable protein, are not necessarily interrelated. In the first approach, the effluent would be treated sufficiently well for irrigation and possibly for release to a stream. Excessive algal production accompanied with inadequate dilution might result in the creation of local nuisance conditions even though the soluble B.O.D. of the pond effluent would be lower than that of the receiving stream. In the second case, the given amount of waste would be stabilized to a desired level with a limited number of algae or else passage of the algae into the effluent is restricted. If, as in the third case, it might be desired to produce a maximum amount of recoverable protein and phosphorus, the design must be entirely different. All of the mineralized nitrates, phosphates, and carbon dioxide of the bottom sediments would have to be brought to the surface. Conversion could then be made to cell material.

Examples of factors which influence the effective stabilization of liquid wastes include: the length of day light, hours of overcast and sunshine, air and waste temperature, wind, rain fall, horizontal and vertical movement of the water in the pond, infiltration, evaporation, seepage, depth of pond, orientation of pond with respect to prevailing winds, and type of microscopic organisms. Accordingly, the oxidation-reduction equilibrium is disturbed during darkness or when scum formations occur. During periods of ineffective light penetration, the respiratory demands of algae and bacteria may reduce the oxygen concentration to critical levels. Furthermore, sudden temperature changes may also reduce the oxygen concentration to critical levels. Temperature changes may not only drastically change the bacterial

3. "Algae Research on Oxidation Ponds," by C. E. Renn, American Journal of Public Health, Vol. 44, 1954, pp. 631-34.
4. "Photosynthesis in Sewage Treatment," by H. B. Gotaas, and W. J. Oswald, ASCE Proceedings-Separate No. 686, 1955.

fermentation rates, dissolved oxygen capacity of the stream, but may also necessitate a change in the algal culture. The migration of algae in the vertical direction makes it difficult to predict with any degree of certainty the depth to which useable light energy will penetrate. Excessive wave action near the effluent end may result in the mixing of bottom sediments and may be responsible for excessive or unnecessary algal blooms. Wave action in other sections of the pond, however, is considered beneficial. The discharge of dense, algal suspensions to a stream under some conditions may not be desirable; for in periods of prolonged darkness the dissolved oxygen concentration of the mixture may approach zero. From the design standpoint, such basic data as general seasonal changes, geographical conditions, evaporation, rainfall, and the mean time the sun is above the horizon have been determined with a fair degree of accuracy. On the other hand, variables such as a slight change in nutrient distribution and environmental conditions can produce unnecessary algal blooms which might affect the desired end results.

The choice of pond capacity, provision for recirculation, inlet and outlet structures, dike arrangements, hydraulic and organic loading are other factors which are of fundamental interest to the engineer. Experiments are herein described and some possible relationships to larger ponds are discussed in respect to an overall energy concept. Oxidation-reduction rates are related to both the total heat energy and the available light energy, and as such these concepts are briefly discussed.

Energy Budget Relationships

The rate of biological reproduction, the generic predominance and the utilization of nutrient are related in part to the heat, light and food energy-budget of the stabilization pond. Also of importance are the total insulating properties of the pond, and the total radiation as affected by the hours of sunshine, overcast, darkness, water temperature, and air temperature. For example, the energy budget is in part reflected as temperature, and therefore is also related to the dissolved oxygen capacities of the pond. Light energy is utilized by algae for the production of oxygen and the maintenance of the oxygen reservoir. High energy chemical compounds useable as food substances are likewise important. Complex carbon, phosphorus and other compounds represent potential cell substance.

Heat Energy

The heat budget, as generally applied to storage reservoirs, is written as:

$$E_s - E_r - E_b - E_h - E_e + E_v - E_g = E,$$

in which E_s is the solar radiation incident to the water surface; E_r is the reflected solar radiation; E_b is the net energy lost by the body of water through the exchange of long-wave radiation between the atmosphere and body of water; E_h is the energy conducted from the body of water to the atmosphere as sensible heat; E_e is the energy utilized for evaporation; E_v is the net energy advected by the sewage flow into the body of water; E_g is the net energy conducted from the body of water to the ground surfaces; and E is equal to the resultant energy stored in the body of water. Conduction of

energy due to chemical and biological processes and transformation of kinetic energy into thermal energy are generally neglected. The heat supplied by solar radiation and the heat absorbed by the ground are probably the most important factors in the foregoing heat budget.

The effects of ground cooling, night cooling and limited evaporation were demonstrated by data obtained from a pilot plant constructed on the surface of the ground. In this plant the waste was heated by direct exposure to sunshine. Results are shown in Table 1. The concrete pond was 35 feet long, 4 feet wide and 4 feet deep. The liquid depth was maintained at the 2-foot level. Since wind movement was considerably limited as a result of physical restrictions, surface evaporation was materially reduced. Based upon evaporative losses of 15 per cent and upon complete mixing, it was computed that the pond temperature could have been reduced about 2 deg. F. As compared to the ground temperature of about 70 deg. F., evaporative heat losses are negligible. Moreover, the ground temperature of the Central Texas Area is about equal to the mean-annual temperature. Myers et. al. likewise have shown that the bottom of a pond having a depth of 3 to 4 feet will remain near the mean-annual temperature.[5] In Texas the mean-annual temperatures range from a high of 75 deg. F. in Brownsville to a low of 58 deg. F. in Dalhart.

TABLE I

POND COOLING

LOCATION	DAY TEMPERATURES DEG. F. MIN. AV.	MAX. AV.
Influent	86	97
Pond Center - 6" below surface	80	87
Pond Bottom (approx.)	70	75
Air Temp. (night min. av. 74)	86	97

Evaporation also helps to maintain a fairly uniform temperature, but the net effect is primarily to increase detention time. All other factors remaining the same, evaporation could control the design. During 1954 the total of evaporative losses in Austin, Texas, was 91.35 inches. The range extended from a minimum of 3.38 inches in January to a maximum of 12.58 inches in July. Totals for the state ranged from 120 inches in the southwestern area to 56 inches in the southeastern section. Evaporation in ponds for central and west Texas, assuming a 20-day detention and a depth of 4 feet, would have evaporative losses ranging from 25 per cent in the summer to 6 per cent in the winter. A proportionate increase in B.O.D. loading may be used for the higher losses.

Light Energy

Available light is the key in stalilization ponds and capacities must be

5. "Observations on Texas Sewage Lagoons," by J. Myers et. al., Report to the Bur. of San. Eng., Tex. State Dept. of Health, 1948.

designed accordingly. Sunlight is therefore necessary in an algal-bacterial system but, paradoxically, algae can utilize in photosynthesis only a limited amount of light energy at a time. For example, Myers has estimated that the upper limit of intensity at which sunlight is utilized with full efficiency by Chlorella pyrenoidosa is 400 f.c.[6] This saturation effect imposes a serious limitation on the utilization of light energy by algae, since the incident sunlight intensity in the middle hours of the day is at least 800 f.c. in temperate latitudes. In a homogeneous culture that is deep and dense enough to absorb practically all of the light, mutual shading of the cells will result in a gradation of light below the surface that offsets this sunlight limitation to some degree. It has been shown at the Stanford Research Institute that the light intensity in a Chlorella culture decreases with concentration and depth in accordance with the Beer-Lambert law:[7]

$$I = I_o e^{-kcd}$$

where I_o is the original intensity of radiation, and I is the intensity subsequent to passage through a medium with absorption coefficient k, concentration c and depth d. Burlew has pointed out that there will be a depth at which the light intensity just equals the saturation intensity I_s.[8] In addition, all the light that penetrates to greater depths will be utilized with maximum efficiency. At lesser depths only the fraction I_s/I_o of the original light intensity will be utilized. Burlew has further indicated that the fraction f of the light energy that is utilized in photosysthesis by an algal culture is given by the equation:

$$f = \frac{I_s}{I_o} \left(\ln \frac{I_o}{I_s} + 1\right)$$

Taking 8000 f.c. as I_o and 400 f.c. as I_s the value of f is 0.20, which means that only 20 per cent of the energy in the visible spectrum of the sunlight at 8000 f.c. can be utilized photosynthetically, the rest being added to the heat budget. When the incident light intensity is equal to the saturation light intensity, f is equal to 1. It should be noted, however, that the light-saturation effect limits a twenty-fold increase in the incident energy to only a fourfold increase in the amount that may be utilized by the algae in a homogeneous culture.

From the viewpoint of design, the problem is not so simple. The algal and bacterial cultures in oxidation ponds are not homogeneous, occlusion of light by scum formation may occur in varying degrees, density stratification of the liquid medium occurs whenever heat flow is from top to bottom; and apparently some types of algal cells will migrate and accumulate at a certain depth in order to better utilize the light energy.

6. "Physiology of the Algae" by J. Myers, Annual Review of Microbiology, Vol. 5, 1951, pp. 157-180.
7. "Development of Chlorella Culture Process," Stanford Research Institute, Final Report on SRI Proj. No. 191, 1950.
8. "Algal Culture from Laboratory to Pilot Plant," by J. S. Burlew, Carnegie Institute of Washington, Pub. 600, 1953.

Experimental Observations

Light, heat and food energy effects have been discussed; and tests, designed to keep these factors constant, were initiated. These controlled experiments also aided in evaluating some of the other factors which are more easily controlled in practice. Numerous experiments involving different wastes, radioactive tracers, light, temperature, surface agitation, depth, recirculation, detention and organic loading have been performed and studied. Analytical information obtained during these tests included pH, B.O.D., NO_3-N, NH_3-N, Org. -N, P, dissolved oxygen, solids, turbidity, temperature, types of algae, optical density, numbers of algae, evaporative losses and material balances.

The pond equipment used in the tests consisted of both laboratory and pilot-plant capacities. Rectangular aquaria having capacities of 45 liters, depths of 1 foot and baffles were used in the inside as well as the outside tests. Two pilot plants were used, one a rectangular unit having a capacity of 5,000 gallons; the other a series-connected system of three basins having a total capacity of 3,600 gallons.

The laboratory units and the ponds were the same in many respects. Although lighting and depth were both reduced in the laboratory, they were maintained approximately equal as compared to outside light and depth conditions. The laboratory units were held at a 1-foot depth and surface lighting was controlled at 500 f.c., while the ponds were controlled at approximately the 2-foot depth and the outside lighting was 5,000 f.c. for a relatively large portion of the time. Comparisons were made with 2-foot depths because at the time of the laboratory experiments more complete information was available with this particular depth. Present experiments are correlating various depth characteristics. Room temperatures and outside temperatures were virtually the same. The vertical temperature gradient that existed in the different ponds was the major difference. The top to bottom temperature differential in the laboratory units was about 6 deg. F. while that of the ponds was about 12 deg. F. This is an important difference considering that the rate of bacterial reproduction is a function of temperature, and that the capacity of the oxygen reservoir is reduced with the increased temperature.

Sunlight

To illustrate the rapid reduction in B.O.D., two sets of model ponds, each having three aquaria in series, were used. The experiment was performed in sunlight near the pilot-plant units. The average influent B.O.D. in Test 1, Table 2, was 230 mg./l. The influent was a synthetic waste having many characteristics of domestic sewage.[9] The B.O.D. loadings were 515,257, and 172 Lb./Ac./D. for volumes having detention periods of 3.3, 6.7 and 10.0 days respectively. These detention periods were based on influent rates and were equivalent to the sum of the detentions in the three aquaria. B.O.D. removal ranged from 85 to 98 per cent. The remaining B.O.D. of 8 mg./l. in the unfiltered effluent was partially due to algal cells. B.O.D. loadings in pilot-

9. "Studies of Sewage Purification. IV. Biochemical Oxidation by Sludges Developed by Pure Cultures of Bacteria Isolated from Activated Sludge," by C. T. Butterfield, C. C. Ruchhoft, and P. D. McNamee, Sewage Works Journal, Vol. 9, No. 2, March 1937, p. 178.

plants with settled sewage provided data similar to that described above.

By comparison the per cent removal for Test 2 was not as large although the detention time was doubled. A considerable increase in algal population occurred. Increased algal production is also reflected in increased pH. This conversion of cell matter from inorganics is a very important point which is frequently overlooked. Even though the complex organic matter has been reduced, the sum total energy which is available has not changed. In some cases the B.O.D. could conceivably be as great in the effluent as in the influent, although changed in composition.

This algae control problem should also be a design consideration. If the water is to be used for irrigation then algae matter little. If the quantity of pond effluent is high in relation to the receiving stream flow, however, then the system should include a predator section or algal-removal device. Algal-bacterial symbiosis being the first stage treatment and protozoa-fish stabilization the second stage. Such practices are now used in Sweden.[10] Systematic harvesting of the fish would, of course, be necessary.

Artificial Light

An indication of attainable loadings was secured from sewage-fed laboratory units operated under controlled lighting. The settled sewage had an average B.O.D. of 310 mg./l., and surface lighting produced an intensity of 500 f.c. The average maximum and minimum air temperatures were 77 and 71 deg. F. while the aquaria temperatures varied between 77 and 86 deg. F. Lights were turned on and off at 0800 hours and 2100 hours respectively. The detention in days and loadings in Lbs./Ac./D for the three models (A, B, C) were 3.4 - 245, 5.3 - 152 and 11.2 - 70 respectively. Each model consisted of two aquaria, for example, A_1 and A_2. Loads are based on the mean influent rates. The average B.O.D. removals were 91, 93, and 96 per cent for Models A, B and C. Filtration of suspended matter from effluents further removed 25 per cent of the remaining B.O.D.

Material balances with respect to nitrogen and phosphorus indicated some losses to the atmosphere. An audit of the data to account for total, fixed and volatile solids demonstrated: (a) the volatile solids entering the system were not all recoverable due to conversion of these organics to simpler and more volatile compounds, (b) the attrition of total solids corresponded to reduction in the quantity of volatile solids, and (c) almost 100 per cent of the fixed solids were recovered.

The results of a dissolved oxygen survey, Fig. 1, indicate the limit in B.O.D. loading as related to available oxygen. It should be recalled that the conditions for this experiment were approximately equivalent to summer conditions in a pond 2 feet deep, and located in the central Texas area. No measurable D.O. was detected in ponds A_1 and A_2. A few algae were present particularly near the surface and this aerobic surface film apparently controlled the odors. The degree of stability is partially indicated by the high pH. The pH remained above 7.2 in the first tank and was near 8 in the final effluent. This indicates some algal activity even though the liquid was void of D.O. The upper and middle strata of the B_2 aquarium were fairly well oxygenated during the lighted hours. The pH range for B_1 was 7.4 to 7.9, and

10. "Oxidation Ponds in Sweden," by M. Wennstrom, Read before the Royal Physiographic Society, Feb. 9, 1955.

for B_2 was 7.5 to 8.1. In model C, oxygenation was further developed and the increase in algal activity was again indicated by an increase in the pH of the final effluent to 8.4.

Recirculation

Three series-connected model units were assembled to test recirculation effects. Light, temperature, waste loading, and surface agitation were essentially the same for each of the three models. The light, 500 f.c. at the surface, was kept on for 13 hours during each day. Air temperatures ranged between 86 and 97 deg. F. Recirculation was from the effluent end of the second aquarium to the influent pipe of the first aquarium, (Fig. 2). The models, designated as A, B, and C, were provided respectively with recirculation ratios of 9:1, 4:1, and 1:1, based upon influent flows. Although the detention ranged from 5 to 6 days, material balances indicated that about 23 days of operation were required before the systems reached equilibria.

Compared to single pass treatment, recirculation increased the B.O.D. reduction. The synthetic sewage fed into each model had a B.O.D. of 200 mg./l.[11] The mean B.O.D. values of the effluents, after the models had become equilibrated, were A = 6 mg./l, B = 4 mg./l, and C = 4 mg./l. It should be noted that, in addition to the exceptionally good B.O.D. removal provided by recirculation, the higher recirculation ratios did not materially increase the B.O.D. reduction.

During the experimental period of 70 days, graphs of B.O.D. and nitrogen constituents in the effluents showed two distinct phases in the utilization of nutrient materials. Correlation of the B.O.D. and nitrogen parameters may be noted by comparing Figs. 2, 3, and 5. Utilization of NH_3 - N (Fig. 3) was quite pronounced during the establishment phase of each culture. Also Org. - N (Fig. 4) was not nearly as well utilized as NH_3 - N during this early phase, while the reverse was true after equilibria were reached in the three systems. As expected, however, soluble nitrogen constituents were more readily utilized than insoluble compounds. At the end of the experimental run, analyses of the aquatic culture remaining in each model pond showed a considerable build-up or retention of nitrogeneous compounds; on the other hand, a material balance for total nitrogen showed recoveries of only 70 to 80 per cent. It is postulated that denitrification occurred from time to time with resultant effusion of N_2 and NH_3 to the atmosphere. As compared to low recirculation ratios, this loss of nitrogen was greater for single pass treatment. However, high recirculation ratios did not increase the recovery of nitrogen above that obtainable with low recirculation values.

Total and soluble phosphorus concentrations in the effluents indicated that phosphorus passed through the models at a uniform rate. Material balances indicated further that 79, 82, and 94 per cent recovery was obtained in A, B, and C models respectively.

Compared with single pass treatment, the use of recirculation decreased the loss of phosphorus to the atmosphere (possibly as hydrogen phosphide). It must be noted similarly that an increase in recirculation caused a greater loss of phosphorus from the liquid system.

11. "Oxidation Ponds, Waste Treatment Studies," by E. F. Gloyna, E. R. Herman, and W. R. Dryman, Technical Report No. 2, The University of Texas, Austin, Texas, June 1, 1955.

The total solids fed into each model was 458 mg./l., of which 37 per cent was fixed matter and 63 per cent volatile matter. At the beginning of the experimental run from 80 to 95 per cent of the total solids appeared in the effluents; after 20 days of continuous operation the effluent levels of total solids had dropped to less than 50 per cent of the mean influent value, and they remained in this region for the rest of the run. Accompanying this reduction in solids was a shift in the relative proportions of fixed and volatile matter so that the effluents now contained about 25 per cent volatile matter and 75 per cent fixed matter. Thus material balances for the fixed solids through each system indicated recoveries of 103, 94, and 111 per cent for models A, B, and C respectively. The deviation from 100 per cent may be attributed to experimental error or, possibly in the case of high values, to the formation of carbonates by absorption of carbon dioxide from the atmosphere.

Results of a D.O. survey are depicted in Figs. 6, 7 and 8. In model C, with a recirculation ratio of 2, no measureable D.O. was produced in aquarium C_1. Oxygen liberated by the algal cells evidently was utilized immediately in bacterial oxidation. In the second aquarium of this system, C_2, the top and middle strata were well oxygenated during the "lighted" hours; and, although the concentration of D.O. in the bottom samples was not nearly as high, there was at least 14 per cent of oxygen saturation present even at the end of the "night." It is especially significant that the D.O. in the top, middle and bottom strata became equal shortly after the lights were turned off and maintained a positive oxygen reserve throughout the dark hours. As noted in similar experiments in which recirculation was not provided, the entire contents of the second aquarium were devoid of D.O. during most of the "night" period.

A recirculation ratio of 4:1 was used in the operation of model B and an interesting phenomenon was observed. Dissolved oxygen from B_2, the second aquarium, was brought around to the first tank with the result that about 50 per cent of oxygen saturation was reached in the top and middle strata during the "afternoon" hours. The bottom stratum, however, remained devoid of D.O. The presence of D.O. in the first aquarium failed to occur when no recirculation was provided, nor did it occur when the recirculation ratio was only 1:1. It should be further noted that during the dark hours the concentrations of the D.O. in top, middle and bottom strata converged to a common level which was appreciably lower than that occurring when the recirculation ratio was 1:1. The gain of D.O. in B_1 seems to have been at the expense of B_2. Due to the disturbance of the hydraulic pattern by sampling beyond the influent rate, no D.O. appeared in the first tank when the survey was extended beyond a 24-hour period. Taking the controlled cyclic functions of the model into consideration, it seems only reasonable that the observed "reinforcement" in the cyclic production of oxygen by the algal cells resulted from synchronization of the lighting cycle of one day, detention period of 5 days, and recirculation ratio of 4:1.

Dissolved oxygen distribution in model A, having a recirculation ratio of 9:1, shows a complete reversal of normal conditions relative to the first and second aquaria (Fig. 6). Instead of the normal pattern, in which higher concentrations of D.O. occurred in A_2 than in A_1, there was more D.O. in A_1 than in A_2. In addition to the above, both aquaria were devoid of D.O. during most of the dark portion of the lighting cycle. As in model B, disturbance of the hydraulic flow pattern by the sampling procedure decreased the D.O.

level in A_1 and increased it in A_2. All of the foregoing observations relative to model A tend to substantiate the hypothesis that "reinforcement" in the cyclic production of oxygen by the algal cells is a function of recirculation. This phenomenon will be of importance in the development of design and operation criteria dealing with the acceptance of shock loads and odor control.

Structural Features and Accessories

The successful operation of stabilization ponds depends greatly upon the structural factors and accessories. Some of these features can be analyzed on an experimental basis while others must be based upon engineering judgment. In addition to B.O.D. loading and detention criteria, other important factors include location, vegetation control, embankment erosion, distribution and collection devices, recirculation, and physical arrangement of ponds.

Stabilization ponds, as is true for all biological systems, are potential odor producers. Good engineering dictates, therefore, that the ponds should be at least one mile from communities and one-half mile from farm houses. If a pond is properly located, well designed and maintained, odors should be of little trouble. If the soil is porous or the location is in a fissured limestone, thought should be given to the possibility of contaminating a water supply.

Shallow marginal areas and emergent vegetation should be eliminated. Aquatic plants will grow in water less than 2 feet deep and such plants protect mosquito larvae.[12] Some mosquitoes, such as Culex, prefer to breed in polluted waters. Both Culex tarsalis and Culex quinquefasciatus have been incriminated in an outbreak of infectious encephalitis. This is a serious disease and resembles poliomyelitis in some respects. Since mosquito larvae cannot survive without protection, maintaining a pond water depth of 2 feet is the most economical means of combating this problem.

Embankments must not be permitted to erode and thus reduce the marginal water depth. Dikes should also be constructed to facilitate future mowing operations.

The most efficient distribution can be obtained in the influent by use of a perforated manifold. Rapid mixing is paramount; and collection and distribution between series-connected ponds can best be attained by using connecting pipes. Special dike problems, such as erosion around the pipes, must be properly eliminated in the design. Moreover, savings can be made by using a greater number of connecting pipes, thereby obtaining a more uniform distribution and greater effective pond area.

Recirculation may be necessary, especially where strong wastes need to be diluted as soon as possible. Maximum efficiency at least during the daytime may be obtained by recirculating pond water that is oxygen rich and that contains the maximum number of algae. It may be that better results will be obtained at night if the effluent, which should be low in algae cells, is recirculated. Algae undergo respiration at night and it is therefore undesirable to use the available oxygen unnecessarily.

The shape of the ponds is also important. It appears that several units within a pond system give the best results. In this way a gradation of

12. "Malaria Control On Impounded Water," by E. L. Bishop et al., U. S. Gov. Printing Office, Wash., D. C., 1947.

aquatic organisms may be obtained. As recirculation makes the water in each pond homogeneous, series-connected ponds aid in proper distribution. In the first section, where rapid dilution is required, the ideal solution would be a circular pond with center discharge and peripheral collection. Obviously, this is not feasible in a large pond. The alternative solution would be a uniform, transverse discharge along one side of a square or rectangular pond.

Experience gained from existing ponds and laboratory data indicate that the liquid depth should not be greater than 4 feet in the primary area in which intensive bacterial oxidation takes place. This primary area may involve one-third of the overall surface area. The ponds—consisting of the last two-thirds of the total area—can have increased depths, depending upon the loadings.

CONCLUSIONS

1) Waste-stabilization ponds, if properly engineered and operated, will accomplish a high degree of treatment; but each new location requires special engineering attention.

2) Primary consideration for design should include an evaluation of: annual, seasonal and daily variations in light intensity and duration of light; air, ground and waste temperatures; evaporation rates; and seepage rates.

3) The B.O.D. loadings, assuming climatic and geographical conditions such as exist in Austin, Texas, can be 200, 150 or 50 Lb./Ac./D., providing that the effluents are used for irrigation, diluted with adequate stream flows, or discharged into intermittent streams respectively.

4) Recirculation ratios up to 1:1 may be required if unsettled sewage is to be efficiently treated in stabilization ponds. Recirculation should be between the point of maximum algae production and influent during the day; during darkness, however, dilution should be made by recirculating the oxygen-rich effluent, which contains a minimum number of algae.

5) Bank stabilization for erosion control and also marginal vegetation elimination for mosquito control, must be incorporated into the design and maintenance plans.

6) Inlet structures for the first pond should consist of a submerged manifold with multiple outlets. Inlets for additional ponds may be the extensions of outlet pipes from previous ponds.

7) The depth in the first one-third of the overall-pond area should not be over 4 feet. The depth of the secondary sections might be increased depending upon local conditions.

ACKNOWLEDGMENTS

The research described herein was made possible in part by a grant from the Sanitary Engineering Section, The United States Atomic Energy Commission, and assistance from Balcones Research Center, The University of Texas. The authors also wish to acknowledge the assistance of various graduate students of the Sanitary Engineering Research Laboratories.

TABLE 2

AQUARIA - UNCONTROLLED TEMPERATURE AND LIGHTING
(SYNTHETIC SEWAGE, pH = 8.0)
ONE FOOT DEEP

Test	Days of Test	Source	Deten. Days	Turb. mg/l.	Algal Cells/ mx10^3	Influent BOD mg/l.	Influent BOD Lb/Ac/D	pH Eff.	BOD Eff. mg/l.	BOD % Rem.
1	2	3	4	5	6	7	8	9	10	11
1	50	Aguarium No. 1	3.3	-	173	230	515	8.0	34	85
	50	No. 2	6.7	-	131	34	257	8.1	10	96
	50	No. 3	10.0	=	315	10	172	8.7	8	98
2	93	Aguarium No. 1	6.7	570	1200	210	235	9.5	27	87
		No. 2	13.4	560	1500	27	117	10.1	19	91
		No. 3	20.0	640	1100	19	78	10.5	16	93

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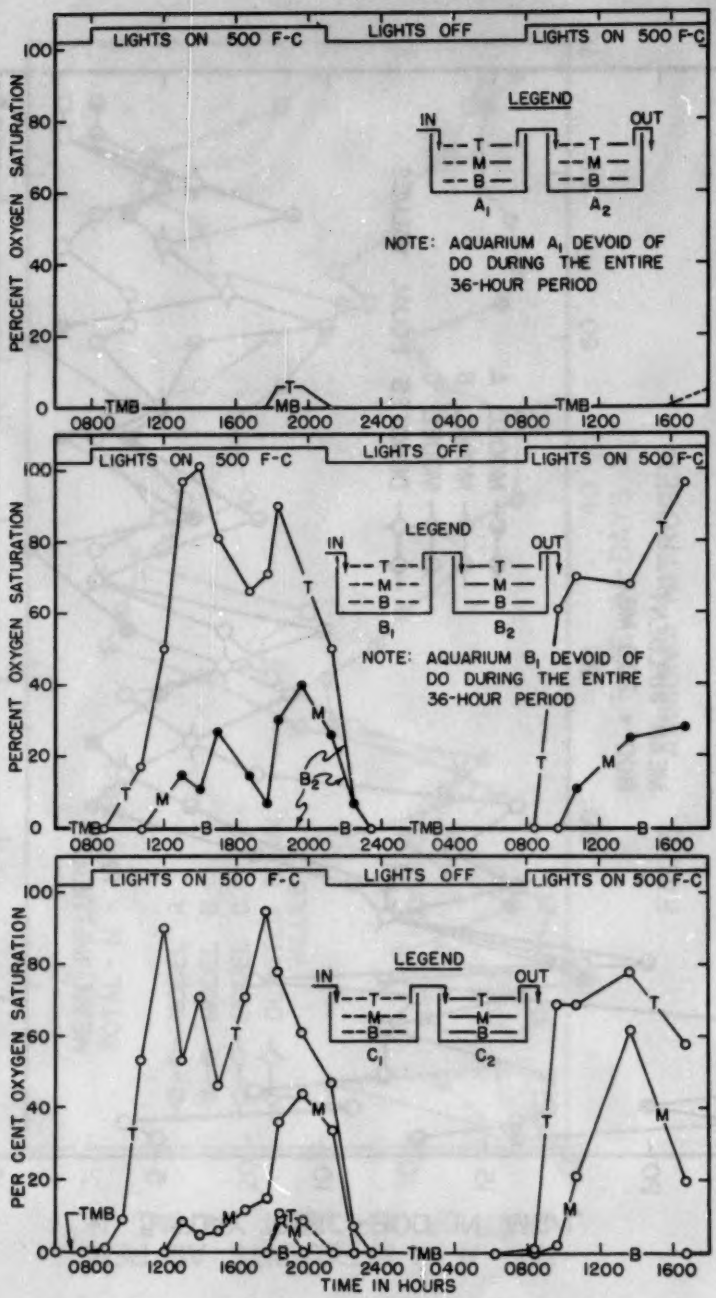

FIG. 1 DISSOLVED OXYGEN SURVEY - MODELS - SEWAGE

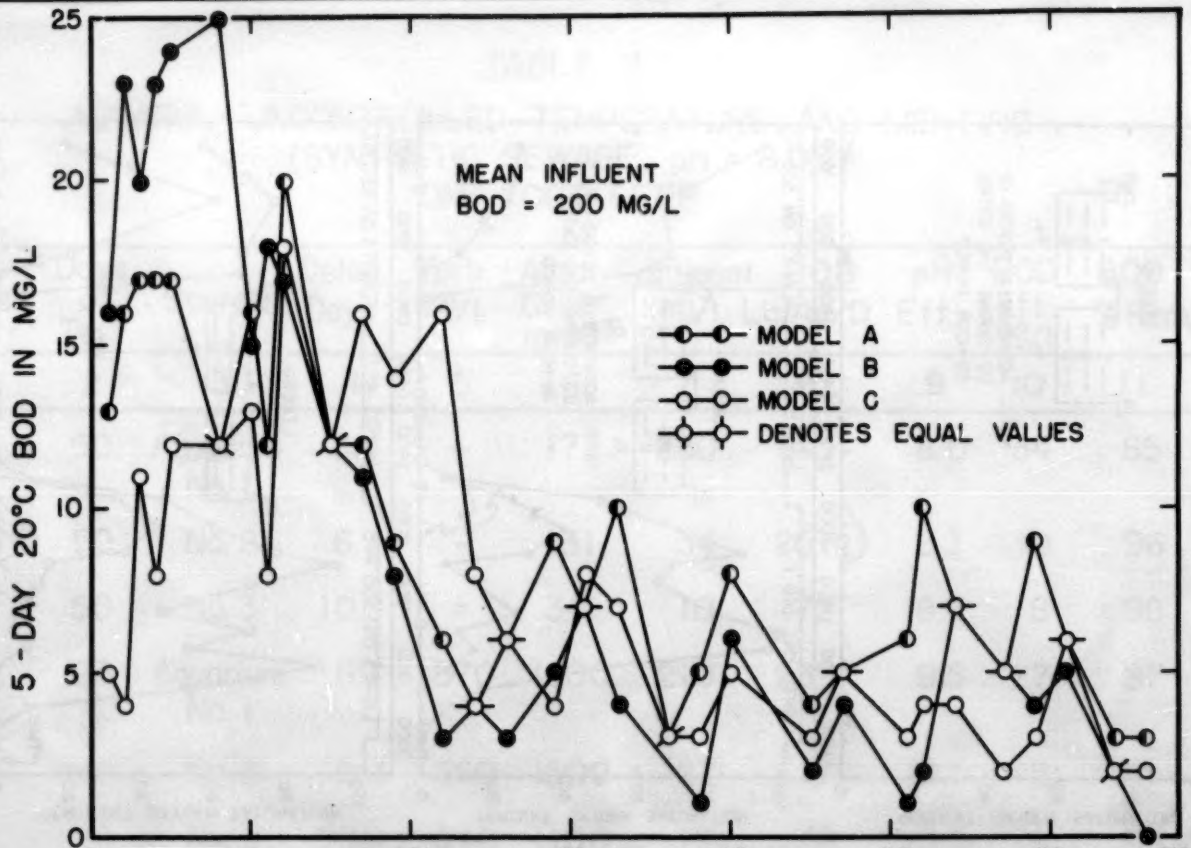

FIG. 2 BOD REMAINING

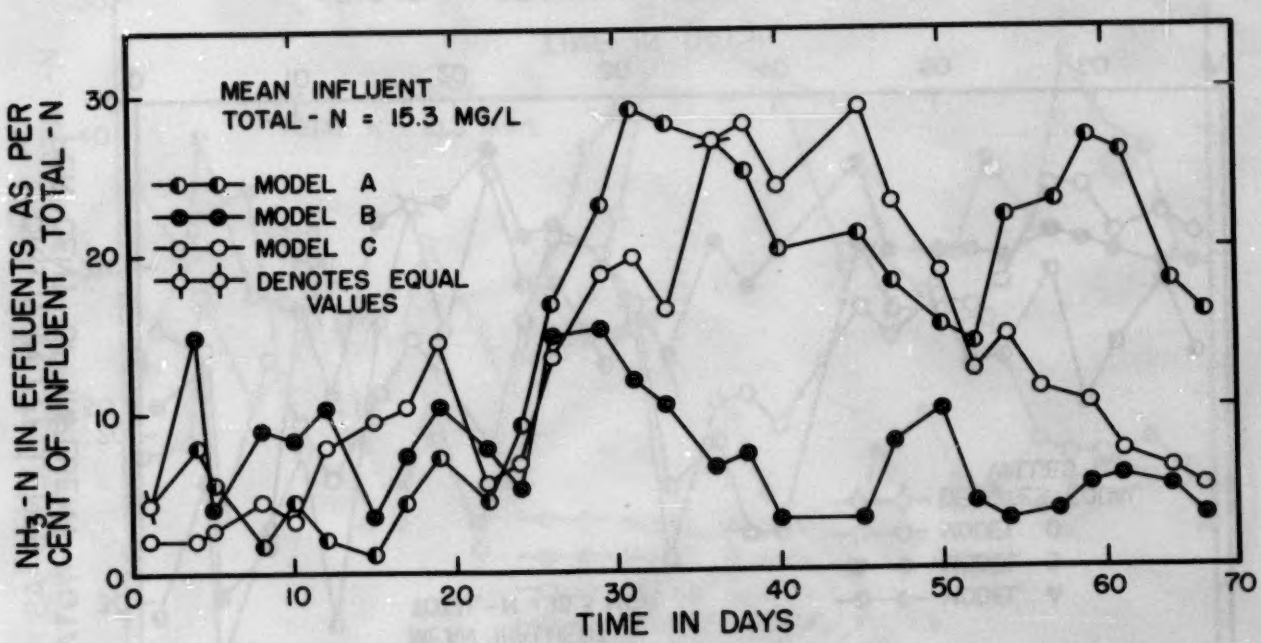

FIG. 3 AMMONIA NITROGEN

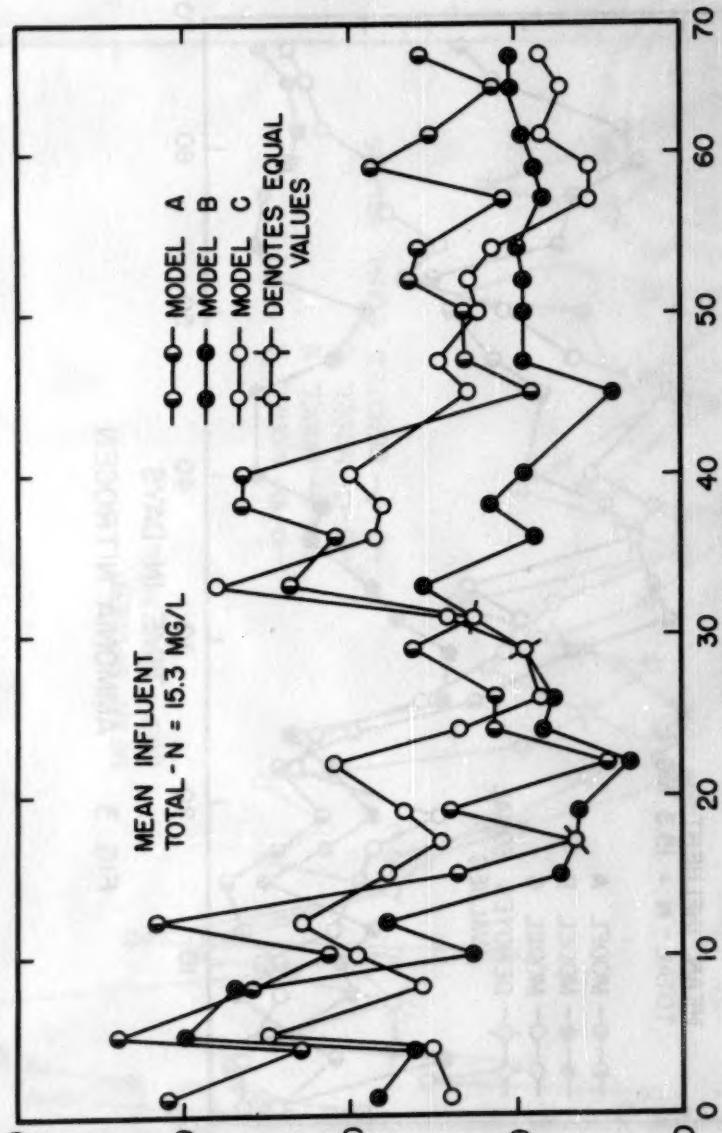

FIG. 4 ORGANIC NITROGEN

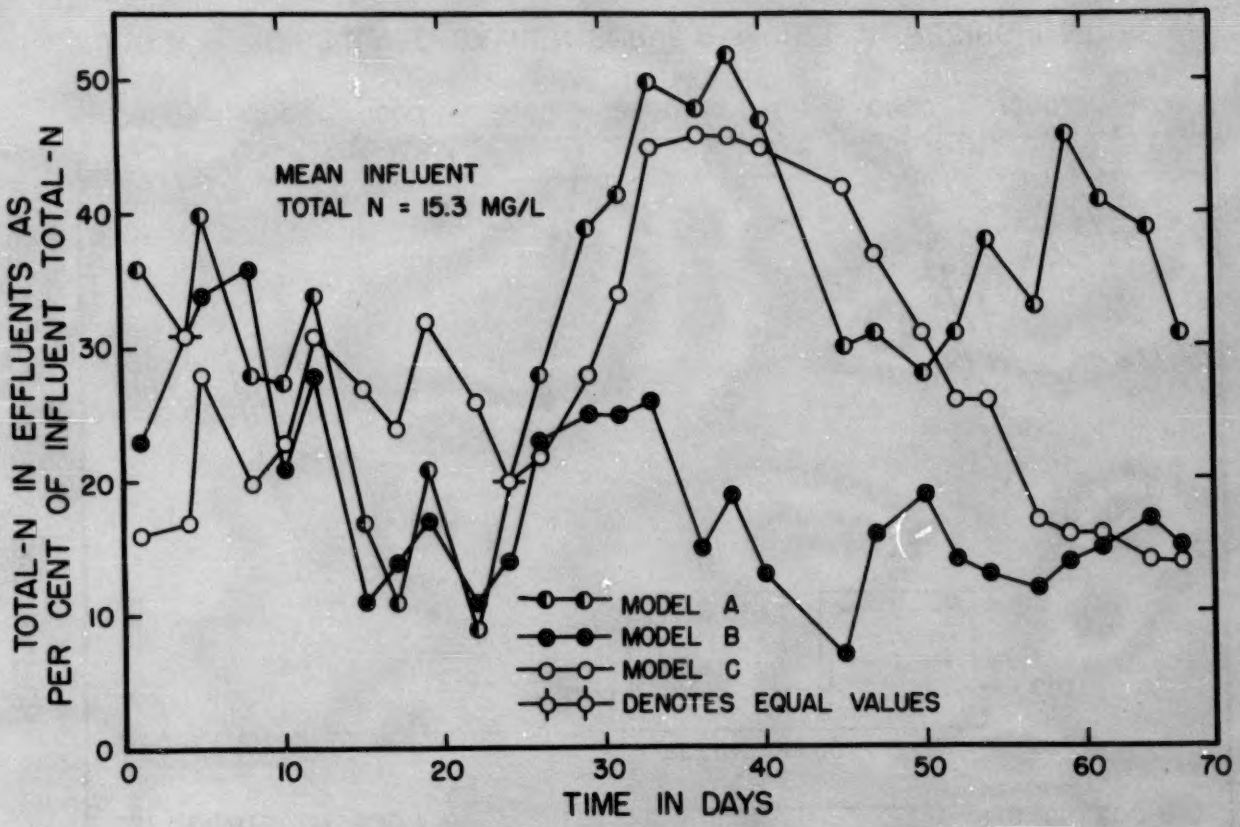

FIG.5 TOTAL NITROGEN

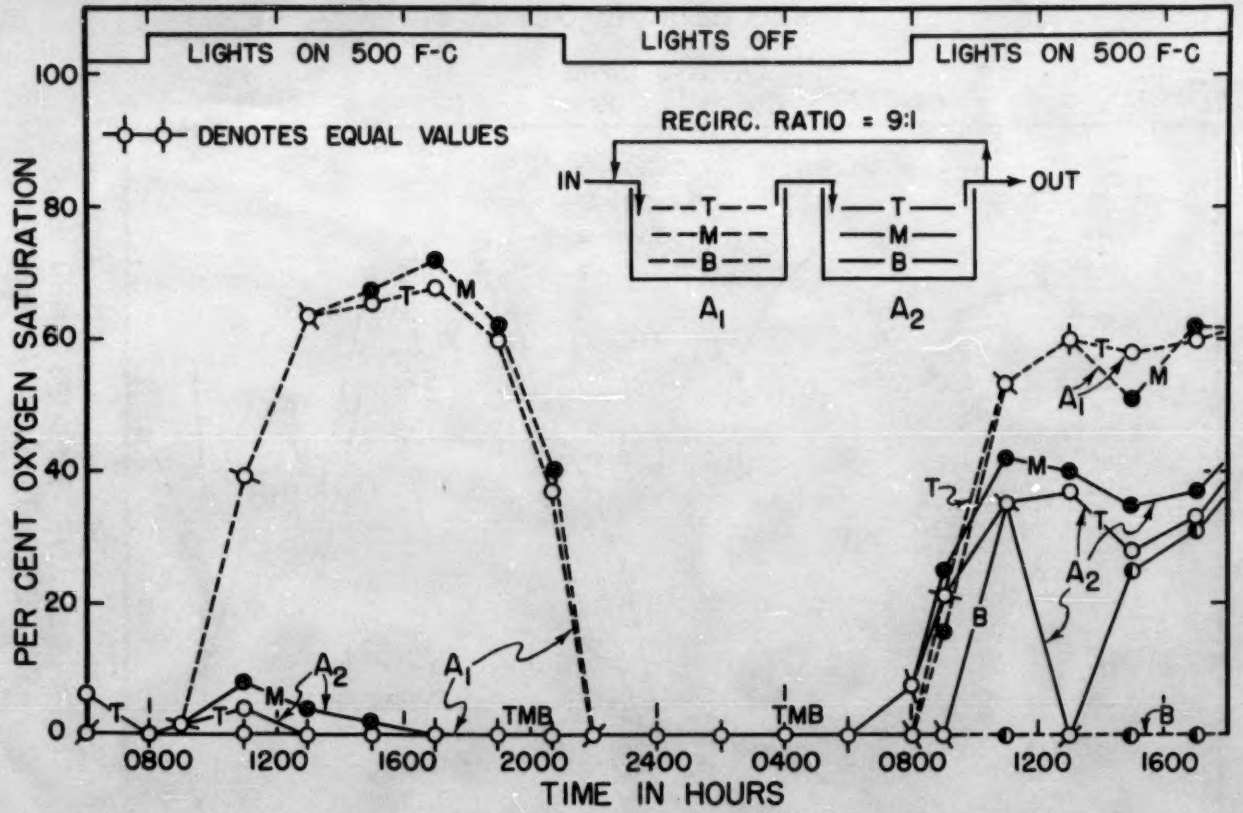

FIG. 6 DISSOLVED OXYGEN SURVEY - MODEL A - RECIRCULATION

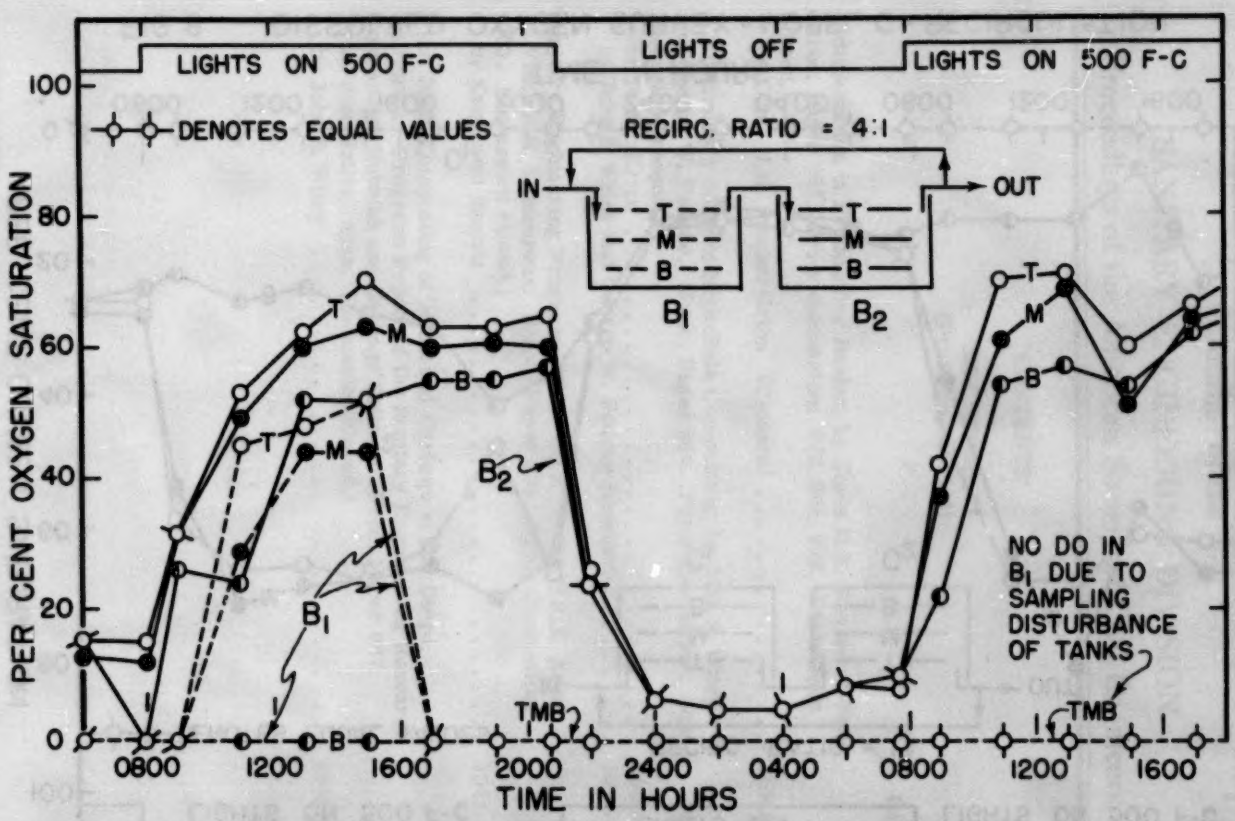

FIG. 7 DISSOLVED OXYGEN SURVEY - MODEL C - RECIRCULATION

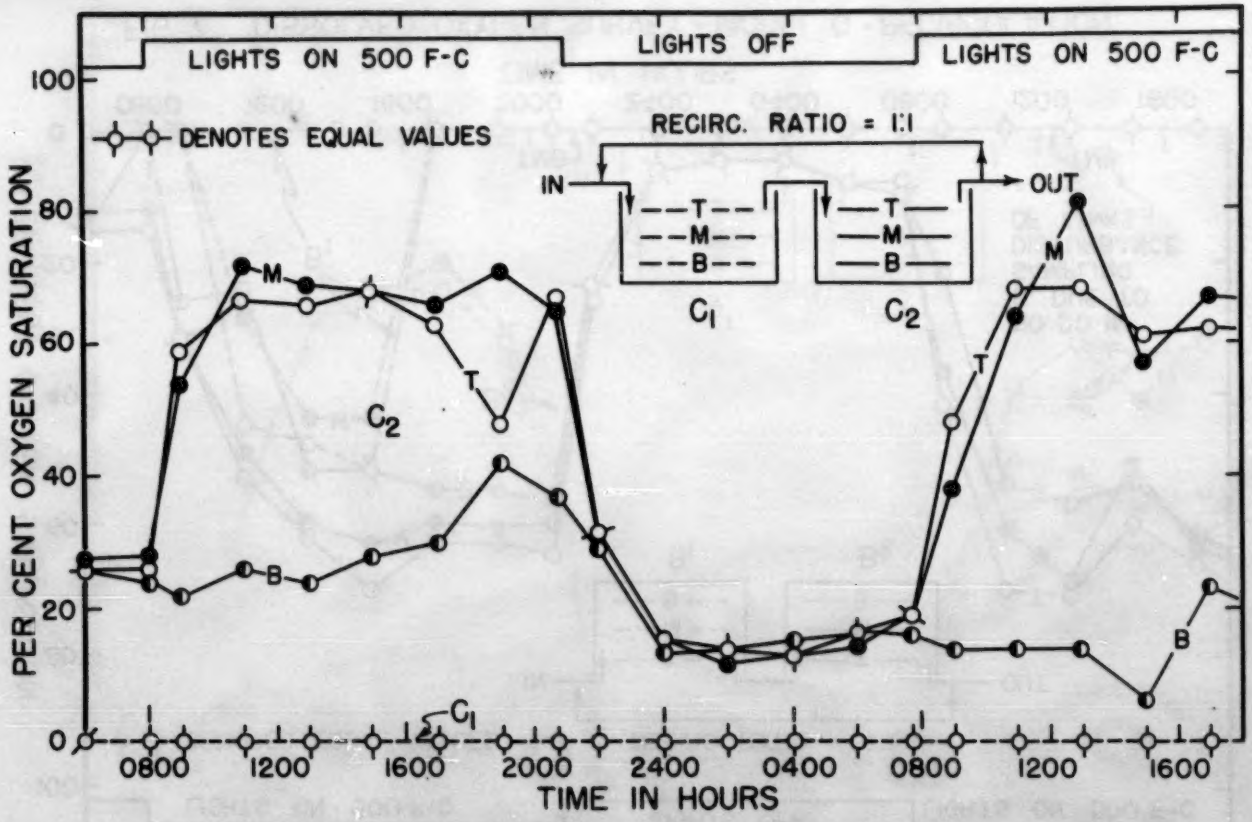

FIG. 8 DISSOLVED OXYGEN SURVEY - MODEL C - RECIRCULATION

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SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

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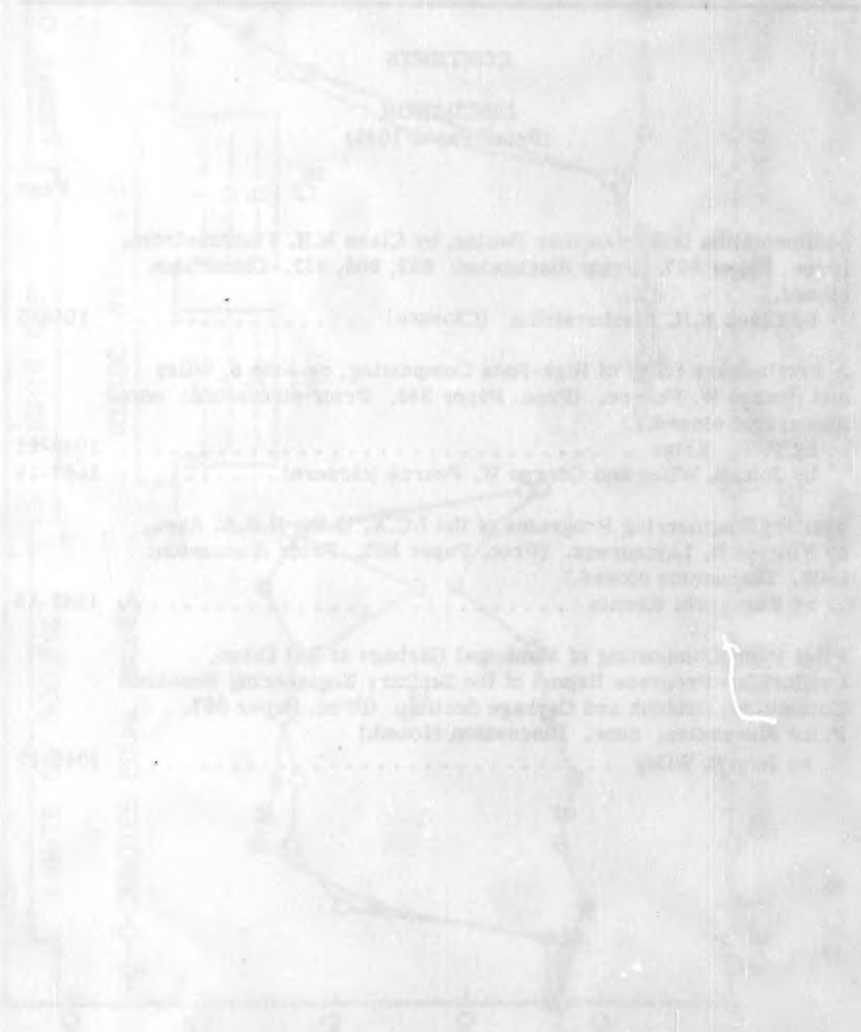


Discussion of
"SEDIMENTATION IN RECTANGULAR BASINS"

by Claes N. Y. Fischerström
(Proc. Paper 687)

FISCHERSTRÖM, CLAES, N. H.,[1,2] M. ASCE.—The writer wishes to thank Mr. McKee for the extensive discussion, but except for the nonexisting correlation between polio and the maximum fuzz of peaches he cannot accept the statements. The reasons will be found below.

Ref. 1.

To begin with, Mr. McKee criticizes the writer for studying only one, or a few factors belonging to a certain group. Mr. McKee refers to some factors as "major factors," and states that it is "a grave omission" not to pay attention to these.

A very great number of factors exert influence upon sedimentation. The main reason for the slow progress in this field, and for the contradictory opinions, even among specialists, seems to be that too many factors are studied at the same time. To study only one, or a few factors while all other factors are considered constant, does not mean that we neglect the influence of the other factors, nor that we deny their paramount effect. By this the writer means that we cannot possibly study, for instance, the influence of different turbulence, if we assume quite different settling characteristics of the settling particles in the various cases; we must, reasonably, only study the influence of different turbulence on the same kind of particles, but we may, perhaps, extend our study to comprise several series, each with a different type of sediment.

It is easier to understand the part played by different factors if we think of the way in which a basin is designed.

1) Let us start with the uncomplicated size-determining factors. Mr. McKee states that "the settling characteristics of the suspension" are paramount. Yes, they certainly are. The overflow rate required can be determined on the basis of a settling analysis curve and the permissible residual. The rate of flow is, however, equally important. The rate of flow and the overflow rate are the size factors, and they determine the smallest possible size of the surface of the basin.

2) A basin with the smallest possible size of the surface will only give the result desired if we have "undisturbed flow." The purpose of the writer's paper is to indicate how we can find the shape or arrangement of such a basin.

---

1. Civ. Engr., Vattenbyggnadsbyrån, Stockholm, Sweden.
2. Mr. Fischerström's closing discussion has appeared in two parts. The first part appeared in "Proceedings" Paper No. 932, April, 1956.

3) There are an unlimited number of basins giving undisturbed flow. Hence, point (2) does not give sufficient conditions, and the design of the basin must, therefore, be determined with due regard to other important factors such as scouring velocity, sludge dewatering, etc. and, of course, to space requirements.

4) It has been assumed that the basin, according to the design finally adopted, will have perfect inlets and outlets. Unfortunately, such things do not exist. These parts must be added to the basin and corrections, necessitated by their ineffectiveness, made (especially in respect of very dirty liquids).

5) Usually there are several basins. Finally, the exact dividing of the flow to the different units is very important.

The paper deals especially with the second step: how to obtain acceptable hydraulic conditions in a basin. It should be quite clear from the above that an undisturbed flow is just as important as the size factors. If a scouring velocity is calculated as a mean velocity, and at the same time the bottom current is several times greater than the mean velocity, this calculation will be as worthless as most others. Controlled flow in a basin is an absolute necessity. Bearing this in mind it becomes obvious that the writer's paper does not deal with "major factors" but "major problems." The main problem is the behaviour of the flow, which depends upon the liquid (concentration of suspended solids and salt, water temperature, etc.) and upon the characteristics of the basin.

The writer is in full agreement with Mr. McKee that it is necessary to pay attention to other factors, but is it really fair to say that the writer has concentrated on one factor without appreciating the others? Quoting from the paper (p. 2): "this paper will deal primarily with the influence of the latter (hydraulic) factors, although their relation to the other factors must be considered;" (p. 14): "The final step would be to control and adjust it to other factors influencing the sedimentation and sludge dewatering," and (p. 27): "going out from a basin with undisturbed flow, it is possible from known properties of the sediment and known local conditions to chose the overflow rate and . . . ."

Ref. 2

The writer does not think that Mr. McKee's comment that the example in Fig. 2 is of academic interest only justifies the conclusion that: "for any sediment with a natural settling velocity distribution, ideal flow will obviously produce the optimum efficiency." If the depth of a basin is continuously decreased, the scouring velocity will soon be reached, and this will occur sooner in a basin with ideal flow than in a basin with, for example, half as great a bottom velocity and a little higher surface velocity. In the writer's opinion, it is very important that it is realized that the assumption of ideal flow in sedimentation is made for the main purpose of simplifying the calculations.

Ref. 3.

Mr. McKee attributes certain superior qualities to the basin provided with multiple surface weirs. The writer has made an analysis showing that this basin, theoretically, can never be better than a plain rectangular basin. Mr. McKee calls the analysis "oversimplified" and the assumptions "a violation against the definition of ideal flow." Well, the basin is not an example of an ideal flow, and if the efficiency is calculated by means of the suggested two-

dimensional flow net, one will arrive at exactly the same result as the writer did. There are too many "opinions" on this matter because of the lack of a mathematical treatise. The proof of this is in the following.

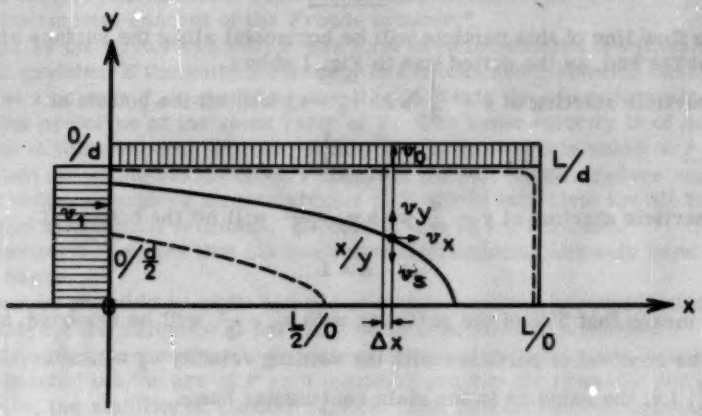


Fig. 1

In a co-ordinate system a particle (x/y) in a basin with evenly distributed inlet flow and evenly distributed surface outlet flow will sink to the bottom via a curved line. To find the equation of this line, let it be assumed that:-

L = length of basin
d = depth of basin
v_1 = inlet velocity
v_o = outlet velocity (overflow rate)
v_s = settling rate of the particle

We will then have the following equations:-

$$v_1 \cdot d = v_o L$$

$$v_x = v_1 \cdot \frac{L - x}{L} = v_o \cdot \frac{L - x}{d} \quad \ldots\ldots (1)$$

$$v_y = v_o \cdot \frac{Y}{d} \quad \ldots\ldots (2)$$

The derivative in x/y

$$y' = \frac{dy}{dx} = \frac{-(v_s - v_y)}{v_x} = \frac{v_o y - v_s d}{v_o (L - x)} \quad \ldots\ldots (3)$$

By integrating

$$d\, v_s\, x + L\, v_o y - v_o\, x\, y + C = 0 \quad \ldots\ldots (4)$$

where C is a constant, the value of which depends only on the starting point of the particle. Assuming x = 0, then

$$C = - v_o L\, y \quad \ldots\ldots (5)$$

If the particle starts close to the surface, $y = d - \Delta y$, $\Delta y \to 0$, we will obtain:-

$$v_S = v_O$$

The flow line of this particle will be horizontal along the surface and vertical at the end, as the dotted line in Fig. 1 shows.

A particle starting at $y = \frac{d}{2}$ with $v_S = v_O$ will hit the bottom at

$$x = \frac{L}{2}$$

A particle starting at $y = \frac{d}{2}$ with $v_S = \frac{v_O}{2}$ will hit the bottom at

$$x = L$$

which means that 50% of the particles with $v_S = \frac{v_O}{2}$ will be removed, and so on. The removal of particles with the settling velocity v_S will always be $d \cdot \frac{V_S}{v_O}$, i.e. the same as in the plain rectangular basin.

The conclusion is that the basin can be as good as a plain rectangular basin, but it is a mistake to believe that this basin is superior to the ideal rectangular basin. In practice, it is inferior because the stability of the basin is poor. In this respect it does not differ very much from the ordinary circular basin.

Ref. 4.

The possibility of using the Reynolds and Froude numbers as controlling factors of the flow in a basin is a more interesting topic. Mr. McKee starts by saying: "the author arrives at sound conclusions (meaning "long narrow basins") by means of incorrect applications of the theories of fluid mechanics." The writer first wishes to draw attention to the fact that his calculations aim at the superiority of the wide shallow basin, if the basin is plain. He has also shown that it is possible to accept any shape by means of longitudinal walls.

An examination of Mr. McKee's arguments reveals that he accepts the Reynolds number as one controlling factor in turbulence. Without making further investigations he draws the conclusion that it is out of the question to keep R below 500 or even 2000, and that, to get a low R, it would, nevertheless, be necessary to have "numerous" walls, even "carried to an extreme;" hence, they are uneconomical and "somewhat impractical." The writer doubts whether this is true.

Mr. McKee has probably arrived at this conclusion because he is unfamiliar with the design of this kind of basin. In Sweden we have, at present, about 25 basins of this type in operation. Many of these basins operate at R = 700 to 1500. The distance between the walls can be about 4 to 6 ft. (vertically and/or horizontally). The load is about 500 gal./sq.ft. per day for each storey of the basin, which means 1500 to 2000 gal./sq.ft. per day in a basin having 3 to 4 storeys.

Perhaps Mr. McKee also overlooked the fact that a very low R is required only when settling very delicate floc. Settling of coarse or heavy material can

take place at a higher R, though we have not yet sufficient experience to state the desirable limiting values of R.

Mr. McKee seems to be dismayed by the use of the Froude number as a stability factor. He mentions some definitions to show that this is "a misuse of the fundamental concept of the Froude number."

As far as the writer can see, F was first used for studies of dynamic similarity in models. If the bottom currents in a model sedimentation basin disappear at a certain F, it should be possible to obtain the same dynamic condition in the prototype at the same value of F. The mean velocity is of no significance in this respect. The above does not mean that this value of F can be taken when designing basins of very different shape. Some readers may think that the writer considers a certain value of F (10^{-5}) sufficient for all types of basins and all kinds of sediment. Of course this is not the case.

The writer reiterates that his fundamental calculation has only been applied to open basins.

It must be possible to make use of the Froude number in closed conduits when studying the influence of part of a stream on the whole stream.

Several prominent hydraulic experts in the U.S. and in Europe have previously pointed out the use of F as a means of judging the dynamic similarity and, hence, the stability of similar basins. Mr. McKee has not suggested any other method of expressing this very important characteristic of a settling basin.

Ref. 5.

Mr. McKee closes his discussion with the following: "There is no question that rectangular settling tanks should be long, narrow and shallow to minimize the effect of inlet and outlet disturbances." In his interesting paper, Proc. Sep. 590, Mr. McKee suggests a length to width ratio of at least 10:1. It seems very necessary to investigate if these disturbances really are so important and unavoidable as to determine this extreme shape of the basin.

The disturbing effects at the inlet could be mainly inlet turbulence and inlet jets. Both the turbulence and kinetic energy will be considerably greater at the inlet pipe than in the basin. The writer has suggested in his paper a "center of gravity" distribution, and preferably the last distributor should consist of multiple baffled orifices. Of course the "distributor" must be designed with due regard to the specific liquid and sediment.

The inlet turbulence causes short lateral movements. The inlet jets may cause movements of large amounts of water through the basin. Large openings at the inlet will always cause such movements. There is no question that the "jets" are of much greater importance than the turbulence as a disturbing factor.

Large scale investigations have shown that it is quite possible to minimize the disturbing effects at the inlet to an extremely short distance from the inlet wall. Fig. 2 shows actual sludge curves in upper channels at the new settling basins at Norrköping waterworks.

The settling plant consists of 6 3-storey basins 43 ft. long and 24 ft. wide. The length to width ratio of the plant is 1:10, but of an individual channel it is 4:1. Load is now 8 M.G.D. corresponding to an actual overflow rate of 450 gal. per sq. ft. per day. Inlet velocity is only 3 in/sec and turbulence about 2,800 in the orifice but 100 when leaving the orifice baffle. Tank Reynolds number is 1,300 and Froude number 10^{-6}. The sludge curves indicate no disturbance

SLUDGE DEPOSIT CURVES
Longitudinal section

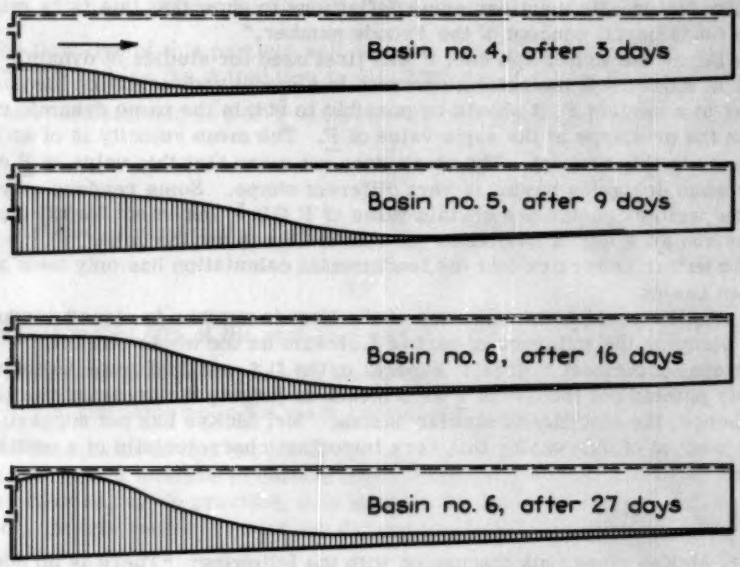


Fig. 2

from the inlet. After 3 days the sludge reaches the lower row of inlet orifices and after 9 days the upper row of orifices. After 27 days both rows are covered, but the upper row will allow the water to enter through small excavations around the orifices, see photo Fig. 3.

The minimum water depth is then only 4 inches (Reynolds number is at that point increased to 2,700), but the sludge layer covers only half the length, and the efficiency is unchanged. The result is obtained with alum-precipitation without activated silica, and can hence be improved.

The above result indicates very clearly that it is not necessary to give a basin a high length:width ratio to minimize the effect of inlet disturbances if the inlets are well designed. Furthermore, a high length to width ratio increases the turbulence and water velocity and the risk of scouring. The main reason to make a basin long and narrow is to increase the (Froude number and) stability, and this may be a most important measure in many cases.

The writer has not discussed the outlet because it is of negligible influence if it is designed as a "diffusion wall" (e.g. by means of a few slots).

Summarizing, the writer believes that hydraulically controlled flow is necessary for a rational study of several factors in sedimentation. Basins with undisturbed flow are already far more economic than the conventional basin and give a much better performance, but it will, no doubt, take a long time before it is accepted that only basins with undisturbed flow should be designed and built. In this connection it should be kept in mind that it took nearly 50 years before the overflow rate concept was generally approved.

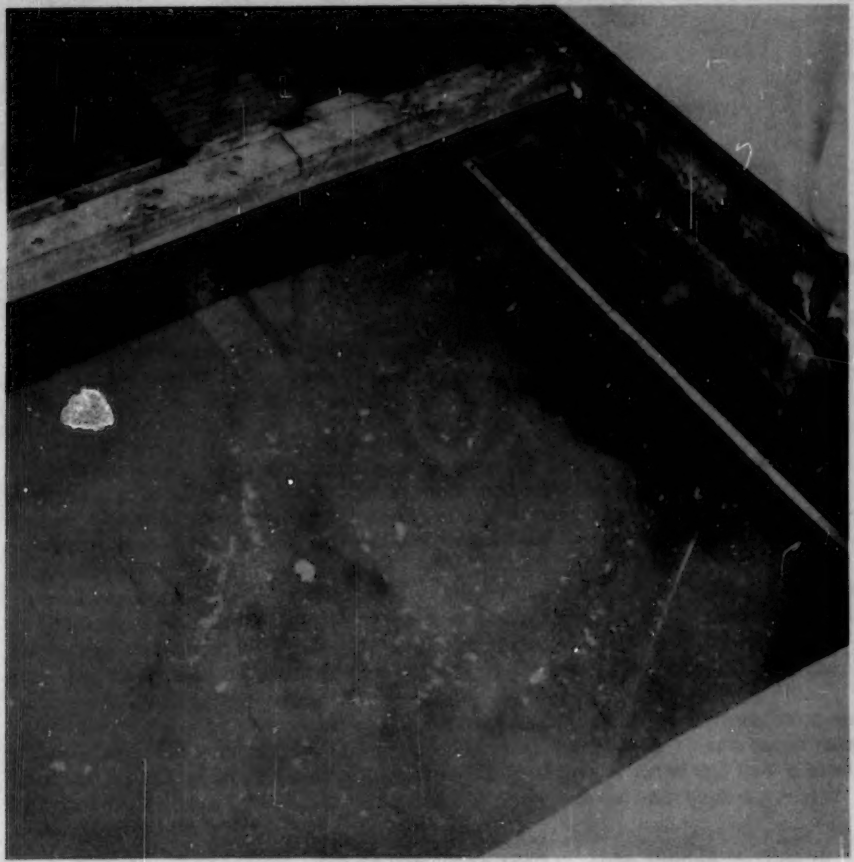
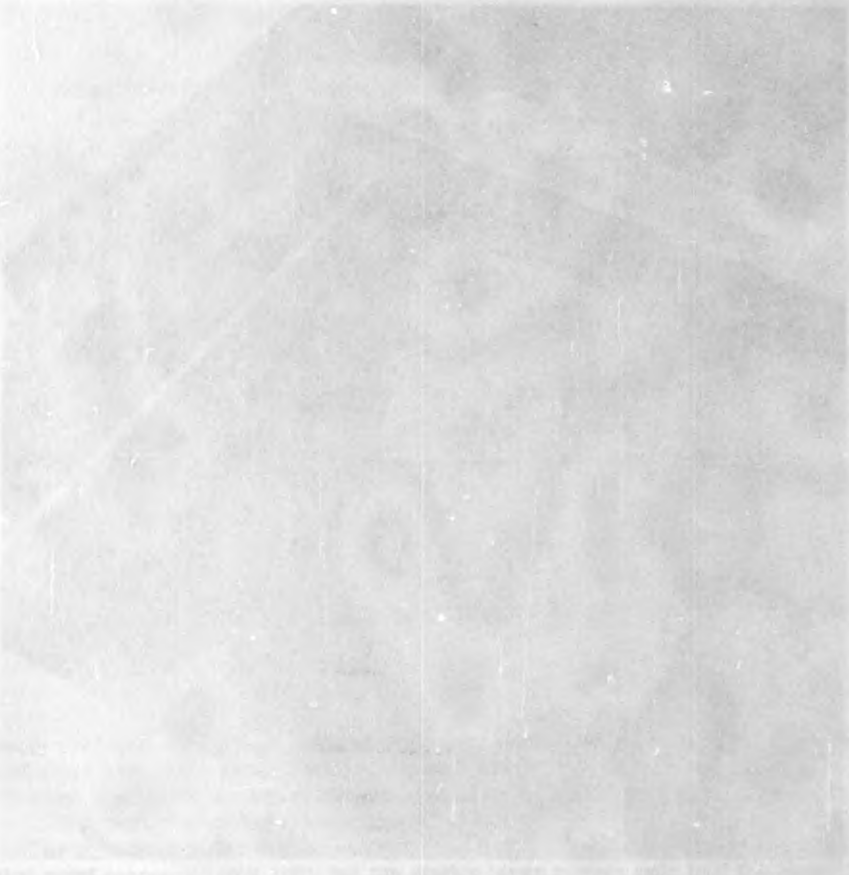


Fig. 3.

Since the sedimentation basin is the purification device which is most widely used, the attitude of leading specialists will be of great importance to future development. At most sedimentation processes considerable savings in money and space can be attained through the application of controlled flow.



Discussion of "A PRELIMINARY STUDY OF HIGH-RATE COMPOSTING"

by John S. Wiley and George W. Pearce
(Proc. Paper 846)

P. R. KRIGE.*—This is a very valuable contribution of fundamental data on the intricacies of composting. The technical information will be most useful in certain phases of work in South Africa in connection with the establishment of a semi-large-scale pilot plant to study more specifically the economics of producing compost from various forms of organic wastes.

Following are brief comments on the above paper:

1) Raw Materials. On page 5 the nature of the raw material used in the experiments is indicated and mention is made of garbage, residential refuse and raw refuse. During the writer's visit to the U.S.A., the impression was gained that the terms "garbage" and "refuse" were not synonymous. It is obvious, however, that this work was confined to the use of "garbage" as the main ingredient.

In the preparation of raw materials resort was made to "throwing out most of the non-compostable items." This procedure would be quite in order when carrying out laboratory studies to evaluate certain chemical and physical factors of composting but can become quite a major operation when translating to commercial installations.

Although mention was made of the addition to the garbage of items such as manure and fish scraps, it would have been most interesting to see more data showing the effect of protein additions on the initial C/N ratio and how a lowering of the ratio might result in higher temperatures but perhaps a greater loss of ammonia.

2) Sampling and testing procedures. The 105° C. oven-drying method of determining moisture leads to substantial errors due to the volatilisation of ammonia resulting in the adoption here of the A.O.A.C. Toluene distillation method. Have the authors considered allowing a bulk test sample to become air-dry, then placing it in an airtight container and doing all the analyses on this air-dry sample, correcting for the residual moisture which is determined by distillation?

3) Typical composting run. On page 9 mention is made of the initial acidic phase. This is no doubt due to a fermentation giving rise to fatty acids. By the third day the acid-producing organisms make way for the thermophilic groups which are able to oxidise the organic acids as fast as they are produced and thereby correct the pH to an alkaline condition.

As regards possible mineralisation of the materials, the volatile solids concentrations indicate that, when working with garbage at the very high figure of 90 to 93 percent combustible, mineralisation during composting is

* Natl. Chemical Research Lab., South African Council for Scientific and Industrial Research, Pretoria, South Africa.

insignificant. In conducting experiments with sewage sludge, grass, refuse, etc., the writer has found that volatile solids decreased from 42 to 25 percent. However, it would be interesting to see the volatile solids in Savannah garbage compost at the age of , say, 6 months and 12 months.

The weight and stoichiometric balances are very informative and well presented.

4) Physical factors for optimum composting. The authors have made a very thorough study of the effect of varying the important factors of rate of stirring, aeration and moisture and arrived at the optimum values for these. Such data are of great value in designing large-scale plants but the writer feels that the authors should have varied the raw materials more, to establish whether these optima hold good for different types of wastes and different C/N ratios.

As is pointed out on page 17, in varying the rate of aeration one must avoid the two extremes, that of lack of oxygen and loss of heat due to over-aeration. It is probable that air control could be very much simplified by installing a continuously indicating CO_2 instrument in the exhaust gases from the unit.

Much more remains to be done to increase technical knowledge of the complex biological mechanisms of composting and it is hoped that further fruitful researches will be reported by the Public Health Service personnel at Savannah.

JOHN S. WILEY, M. ASCE,* and GEORGE W. PEARCE.*—The interpretations of the Savannah data and translation to plant-scale conditions in accordance with South African experience by Mr. Krige are very encouraging. Several nations, especially South Africa and New Zealand, are far in advance of the United States in the conduct of applied research in the field of composting. The authors agree that much more basic and applied research needs to be done before attempting to build even a small pilot plant of the mechanical type in this country.

The authors will attempt to elucidate on the comments by Mr. Krige concerning the work reported from Savannah:

1) Raw materials. "Refuse" normally includes garbage but frequently in the United States separate collections are provided, one for garbage and another for other refuse. The raw material has been termed "garbage and refuse" to show that it contains the all-inclusive domestic refuse. It contains probably only one-third to one-fourth garbage and a high paper content, accounting for the high percentage of volatile matter. That part of the mixture which has been composted contains no grass, tree trimmings, street sweepings, or the like, but does contain paper, trash, and garbage from the home including small amounts of dust, floor sweepings, hair, and similar items of household refuse. The short high-rate composting process hardly alters paper and other cellulosic material and is primarily involved with the garbage portion of the mixture.

The sorting out of non-compostable items will involve major operations in a full-scale plant and not all of the undesirable materials may be economically removed. Salvage, of course, may reduce or cancel the cost of such

* Dept. of Health, Education, and Welfare, Public Health Service, Communicable Disease Center, Technical Development Lab., Savannah, Ga.

sorting; however, all equipment must be constructed to withstand the toughest and hardest materials likely to occur in raw refuse, regardless of the degree of preliminary sorting. Some hard objects may be desirable in a type of mechanical composting unit such as a rotary drum in that they may provide additional grinding or crushing during composting, provide a more open structure for circulation of air, and prevent sticking of wet material to the sides. A 110-gal. rotary drum unit is now being tested at Savannah.

Concerning C/N ratios of raw materials, more experiments are planned in which N is increased in a variety of forms. There must be a sizeable range in C/N, depending a great deal on the types of materials, over which aerobic composting may be accomplished without appreciable loss in ammonia. Some recent observations indicate it may be desirable to keep maximum temperatures down, possibly to prevent self-destruction of desirable organisms.

2) Sampling and testing procedures. It is assumed that Mr. Krige means by "substantial errors due to the volatilisation of ammonia" that these errors are in the determination of nitrogen (either total or ammonia nitrogen) rather than in the determination of moisture. Any error in moisture determination by the oven-drying method as a result of volatilization of constituents other than water is believed to be smaller (say 1.0 percent or less) than the sampling error (occasionally greater than 2.0 percent). The volatilization of ammonia in drying may cause an appreciably low total nitrogen value. At Savannah no nitrogen determinations have been run on either fresh or air-dried samples, only on oven-dried samples.

A few ammonia determinations have been made on the liquid which condenses from the outlet gases by reason of temperature drop from drum to room temperature. The values ranged from none to 600 ppm, with average values for the run varying from 14 to 260 ppm. However, these values should not be considered as quantitative for either the outlet gases or the composting material.

3) Typical composting run. The hypothesis by Mr. Krige concerning the acidic phase of decomposition which occurs initially and at below the normal thermophilic range in Savannah composting runs appears to be sound. A large amount of acid is produced because it was necessary to add 16 percent $Ca(OH)_2$ on a dry weight basis to maintain alkaline conditions, whereas 50 percent agricultural lime (dry basis) had little effect on pH or temperature of the compost. However, if there was sufficient alkaline buffer to prevent acid conditions, there was no temperature lag in the composting process.

The opposite effect also occurs. That is, when the pH starts up and temperature goes rapidly into the thermophilic range, it is difficult to reverse the reaction and maintain an acid condition in the compost. In one run, five additions of glacial acetic acid totalling 12.3 percent of the dry weight of compost resulted in fairly good control of pH below 7.0 but, after the additions were stopped, the pH climbed to above 8.0 on the ninth day.

Regarding mineralization, the authors have only determined volatile solids for 26-30 days total composting time, of which that subsequent to the run took place in open cans. Results in 6 runs, each with 6 units operating, were as follows:

Run	Volatile Solids, Percent (D.W.)		
	Initial (0-day)	End of Run (6-9 days)	After 3d Re-heat (26-30 total days)
25	93.1	91.9	76.2
26	90.7	87.7	75.0
27	90.7	87.7	75.6
28	91.7	88.0	77.3
29	93.6	92.6	84.5
30	92.1	88.8	82.3
Avg.	92.0	89.45	78.5

There is considerably greater reduction in percent volatile solids accomplished during about three weeks in open cans than in one week of composting in the mechanical units. Unfortunately, the actual dry weight or volatile solids losses occurring in the open can composting were not determined.

4) Physical factors for optimum composting. It is agreed that the optimum physical factors for composting must be checked against variations in raw materials. However, this could mean a long study, especially if it involved many of the various types of solid organic wastes produced by various industries. It is hoped that some of the problems associated with municipal refuse composting may be settled first and then those encountered in the composting of various other wastes. The consistency of analyses of the raw refuse from the City of Savannah has been surprising. It is desired to gain enough experience with all types of municipal refuse before recommending mechanical composting universally. In connection with the raw materials, the authors do not place as much emphasis on C/N as do some others in this field.

Estimations of optimum aeration rates were admittedly based on indirect measures, such as speed of composting as indicated by temperature and pH, losses in weights, and moisture and carbon dioxide production. Oxygen utilized during a run was determined by difference between intake and output. Subsequently, it has been learned that moisture production figures have been low and corrections will increase those values as well as the amounts of oxygen utilized. Direct oxygen readings during controlled runs should provide an additional check on the other determinations involving stoichiometric balance, as well as serving as a better criterion for judging optimum aeration rates. (Although it is known that moisture measurements in the outlet gases are probably low, there are indications that there are some compensating errors in the opposite direction so that the material balance estimates are not seriously off. The most reliable data on material balance are those showing the gross loss in dry matter. It is believed that the carbon dioxide estimates are reasonably accurate).

Discussion of
"SANITARY ENGINEERING PROGRAMS OF THE I.C.A.
IN THE N.E.A. AREA"

by Vincent B. Lamoureux
(Proc. Paper 885)

KAZUYOSHI KAWATA,[1] J.M. ASCE.—Mr. Lamoureux in presenting the program of the International Cooperation Administration in the N.E.A. area makes a statement which from the writer's experience seems incorrect. He writes: "It should be noted that rural areas need attention first, not only because the rural villages require improvement but because improvement in the cities will only attract people from these rural areas more rapidly than the municipal authorities can keep pace with the demands to extend and maintain their facilities and utilities." That the rural areas need improvement in living conditions cannot be disputed, but it cannot be argued that improvements in living conditions attract people from the rural areas to the cities. It is something more fundamental than facilities and utilities; it is the matter of earning a living. It can be pointed out that there is an ongoing migration from the rural areas to the urban industrial centers in India even though there is a noticeable absence of improvements in living conditions in many of these industrial centers. In contrast there are a few non-industrial cities enjoying improvements in living conditions that do not attract people from the rural areas. With ever increasing population and with agriculture land being limited more and more people in India who live in the rural areas are looking to the new industries for their livelihood.

Mr. Lamoureux outlines briefly the task in the program of environmental sanitation, that of training sanitarian aides, securing their employment by government, sending them into the rural areas to start the cleanup, and to bring into each community some understanding and undertaking of environmental sanitation and hygiene in the home. These are admirable points if they are based on sound socio-economic study of the people and continued analysis of the situation by a rural life analyst as in the Etawah Project. It might be well to add a rural life analyst or a social worker to the basic public health team. Mr. Albert Mayer, organizer of the Etawah Project, places a very high importance on the rural life analyst. It is on his ground-work and on his analyses that a successful public health program including sanitary engineering projects can be built especially as it involves the habits of the people described by Mr. Lamoureux.

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1. San. Engr., The United Christian Schools, Jullundur City, Punjab, India.

Discussion of
"PILOT PLANT COMPOSTING OF MUNICIPAL GARBAGE AT SAN DIEGO, CALIFORNIA: PROGRESS REPORT OF THE Sanitary Engineering Research Committee, Rubbish and Garbage Section"

(Proc. Paper 887)

WILEY, JOHN S.,[1] M. ASCE.—The recent wide interest in composting in the U. S. is very encouraging because it is the belief of the writer that aerobic thermophilic composting of garbage, refuse, and other wastes will not only prove to be a competitive sanitary method of treatment but will also produce a worthwhile by-product. The investigator is to be congratulated on the comprehensive report[2] which supplied the data for this progress report.

Physical Problems in Composting

Apart from problems in materials handling, grinding, and the like, the major problem in outdoor composting of garbage is the provision of proper physical conditions of the material for optimum aerobic composting. It appears that a combination of such conditions must exist: proper moisture content, adequate particle size distribution, and sufficiently open structure. These conditions are interdependent and may be illustrated by the apple. The apple is a solid; if it is diced, the cubes still are solids with air spaces between them; if it is ground into apple sauce, it becomes a semi-solid with pores filled with liquid instead of air and with insufficiently open structure to drain or to aerate properly.

At San Diego the moisture content of the raw garbage varied from 62.5 to 74.3 percent in the seven piles receiving no additives. The desirable range of moisture was reported[2] to be 40 to 60 percent. Moisture reduction was accomplished by drainage, requiring from 2 to 9 days, or by additions of dry material plus a drainage period. In the latter case, the drainage was still necessary because in no case was sufficient dry material added to reduce initial moisture content below 59.5 percent. A similar moisture problem was experienced at Michigan State University where ground garbage was dewatered for about 3 days in a holding tank before being conveyed to a mechanical composter. Excess water is not the sole source of the trouble in garbage because lipids—fats, oils, greases—act similarly to water in obstructing aeration. Lipids (ether soluble) amount to 8.3 to 12.8 percent of the dry weight of refuse at Savannah and might be 2 to 3 times these values in garbage.

Concerning particle size, it is evident from the San Diego report that

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1. Sr. San. Engr., Dept. of Health, Education, and Welfare, Public Health Service, Communicable Disease Center, Technical Development Labr., Savannah, Ga.
2. Quartly, Eric V. et al. "Municipal Composting on a Pilot Plant Scale." City of San Diego, Dept. of Public Works, Sewage Treatment Division, November 1954.

coarse grinding produced better initial results than fine grinding because the material did not become as soggy. This corresponds to a diced apple as opposed to apple sauce. However, before composting may be sufficiently completed, it appears necessary to reduce particle size to permit the organisms to gain access to the organic matter which is to be decomposed. Some particle size reduction occurs in turning and mixing the composting matter, which probably partly explains multiple peak temperatures observed at San Diego.

Open structure of the material, particularly the availability of pore space and channels for aeration and by-product elimination, appears to be extremely important. Where mechanical composters are not used, aeration between turnings depends on convection and diffusion, neither of which occur well in a soggy or liquid substrate. Garbage, with its high proportion of water, lipids and pulpy matter, has excess liquids, unfavorable particle size distribution, and inferior structure for aerobic decomposition.

The University of California reported[3] successful outdoor composting of refuse in 12 to 21 days, yet the San Diego runs with garbage lasted 30 to 90 days; minimum time for temperature to drop below 40° C. finally was 34 days, maximum was 95 days. Experiences by the Communicable Disease Center at Phoenix and Savannah indicate that refuse will compost readily in six weeks or less in outdoor open bins and in 4 to 11 days in mechanical batch-type laboratory composters. It appears therefore that waste paper and other refuse constituents not only reduce moisture but greatly improve porosity and structure of garbage for composting.

In the San Diego experiments some dry additives were included, but neither paper nor straw were added in excess of 5.6 percent on a dry weight basis. Straw proved to be quite beneficial in small quantities—more so than paper. However, it should not be necessary to add materials which must be purchased when normal domestic refuse has the needed materials, and must also be disposed of. It appears that aerobic garbage composting will not be practical until inexpensive moisture-reducing and open structure-building processes are developed. Mixed refuse may contain only 35 to 50 percent moisture. It is easier to add moisture than to take it away or "dilute" it.

Analyses and Weight Losses

Having obtained the basic report[2] from San Diego, the writer attempted to analyze weight losses attained in the experiments on the basis of total weights and analyses given. Carbon was determined by the formula:

$$\frac{\% \text{ molatile matter}}{1.8} = \% \text{ carbon}$$

and the writer merely reversed the formula to obtain volatile solids content on a dry basis. The computed weights and losses are shown for one pile in table 1.

Obviously the weight losses shown are incompatible. It would be most unusual for the total dry weight to increase 16.8 percent while volatile matter decreases 68.8 percent. Volatile matter in raw garbage amounts to 84 to

3. "Reclamation of Municipal Refuse by Composting." Tech. Bull. No. 9, Sanitary Engineering Research Project, University of California, June 1953.

Table 1: Weights and Losses in Pile III (Pounds).

Constituent	Initial	Final	Gain or Loss	Gain or Loss, %
Total	6800	3160	-3640	-53.5
Moisture	4660	660	-4000	-85.9
Dry Matter	2140	2500	+360	+16.8
Volatile Solids	2020	630	-1390	-68.8
Nitrogen	53.2	26.7	-26.5	-49.8

94 percent of the total dry weight and would not be expected to be less than 50 percent of the final total dry weight. Values in table 1 could also be interpreted to show an increase in ash from 120 pounds to 1870 pounds, or 1750 pounds during composting. The dry weight and volatile solids weight changes shown for Pile III are more widely divergent than for any of the other 11 piles but, in all except Pile XII, the dry weight showed either a gain (3 piles) or a much smaller loss (8 piles) than the loss in volatile matter. Experiments in high-rate composting at Savannah[4] have shown no significant mineralization or increase in total ash during a run (the percent increases but this is due to a decrease in dry weight). There has been no significant difference between weight losses of dry matter and of volatile matter.

A suggested explanation for the above discrepancy is that, in the process of turning (28 or 29 times for Pile III), sufficient soil or sand was picked up with the compost to materially influence the final analyses of the composted matter. Contributing factors may also be the presence of wind-blown dust or settled atmospheric dust in the compost. Such contamination would increase dry weight and ash determinations while decreasing moisture, volatile solids, and nitrogen determinations. No suggestion is made that either sampling was not representative or analyses were inaccurate. It merely appears that 1000 to 1200 pounds of soil or dust became mixed with the compost in Pile III by the 100th day of composting when it was ended.

More plausible weights for Pile III would be final total, dry, and volatile solids weights of 2000, 1340, and 1220 pounds, respectively, showing equal losses in dry and volatile matter of about 800 pounds. Such values show better composting than the investigator reported on the basis of weight losses, poorer than on the basis of constituents percentage-wise. Final percent nitrogen for compost excluding contaminants might be materially increased but it would be hard to estimate the amount of nitrogen weight loss. In Phoenix, when composting was done in piles on the ground, it was found that nitrogen determinations were unreliable because of soil contamination in compost samples.

4. Wiley, John S. and Pearce, George W. "A Preliminary Study of High-Rate Composting." ASCE Proc. Paper 846, December 1955.

Cost Computations

This paper includes a table of costs of composting showing variation from $1.56 to $20.48 per ton of fresh garbage. The report[2] indicates that the cost figures are per ton of final pile weight or compost. Re-computing the values on the basis of raw garbage reduces costs to $0.63 to $6.93 per ton compared to $1.56 to $20.48 per ton, a two-thirds reduction of the values given in the paper.

Initial transferring and piling costs after grinding appear to be excessive. For the most expensive operation (Pile VIII) the total cost was $18.71 for the pile ($20.48/ton of final pile weight) and of this amount $4.90 was for transferring from grinder to pile and forming the pile. This operation was performed by one man with a shovel and wheelbarrow. On the other hand, turning of the pile, accomplished 23 times with a skip loader, cost a total of $0.91, or about $0.04 per turning. The use of mechanical equipment for transferring and piling should reduce the cost from $4.90 to $0.12 to $0.20, the cost of three to five turnings.

The investigator used a garbage composting time of 90 days in determining the area of 25 acres required for a 50-ton per day plant, allowing 100 percent safety factor. With proper raw materials for composting, the time might be reduced to 21 to 30 days. University of California estimates[3] for a 36-ton per day refuse composting operation were 8 to 10 acres of total land area.

Ideally the best municipal arrangement for composting might include the treatment of the following wastes with disposal to market: Combustible refuse, garbage, concentrated or filtered raw sewage sludge, and possibly certain solid industrial wastes. However, several plantscale composters in the United States have failed miserably due to inadequate preliminary basic and applied research and to pilot-scale tests such as this very worthwhile study at San Diego.

Paper 1049 SA 4

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ENGINEERING AND MALARIA ERADICATION[a]

Patrick N. Owens,[b] A.M. ASCE
(Proc. Paper 1049)

SUMMARY

This paper describes the steps leading up to the establishment of programs for the eradication of malaria by the countries and territories of the Americas. The responsibilities of engineers in these programs are outlined, and a typical program is described. Special problems relating to the design of equipment are also discussed.

The official health representatives of the twenty-one countries of the Americas, meeting at the XIV Pan American Sanitary Conference in Santiago, Chile, in October, 1954, resolved to embark upon a program to eradicate the disease, malaria, from the Western Hemisphere. The Pan American Sanitary Bureau was assigned the task of coordinating eradication programs within the Americas, and with providing technical assistance for their successful prosecution. The purpose of this paper is to describe some of the reasons for such a decision by the Pan American Sanitary Conference, and some of the challenges and implications for engineers contained within it.

From apparently the earliest days of colonization of the Americas, malaria has been one of, if not "the" most important disease. Although its origins on the Continent are obscure, Gabaldon[(1)] cites references which describe a disease in the Americas resembling malaria in 1496, and to its probable existence in the Paraguay River basin as early as 1543. It is described as being endemic in the United States by the time of the Revolutionary War, as far north as Pennsylvania[(2)], and it followed the westward course of migration, until a large portion of our country was estimated as endemic by the 1880s[(3)]. Malaria has extended as far north as Canada, and as far south as San Luis, Argentina, lat. 33° S.

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a. Presented at the Winter Meeting, Dallas, Texas, February 13-18, 1956.

b. Sanitary Engr., Coordinating Office of the Malaria Eradication Programs, Pan American San. Bureau, Mexico City, Mexico

The limits of malaria have never been static, however, but ebb and flow as migration, opening of new lands, sanitation, or improved economic conditions affect the circumstances required for maintaining transmission. Malaria had receded from its northernmost penetration by the end of World War I, so that it was possible for Sir William Osler to predict that no serious problem would occur in Canada on the repatriation of veterans from malarious areas(4). In the United States, by the end of 1945, only five endemic spots were recognized, distributed in South Carolina;(coastal counties), one county of Georgia; nine counties of the upper Mississippi delta in Arkansas, Louisiana and Mississippi; two adjacent counties on the Red River in Texas and Arkansas; and the lower Rio Grande Valley(2).

The economic importance of malaria has been estimated for several countries. In 1938 Williams(3) estimated the yearly cost of sickness for the United States to be about $51,000,000 and the economic loss to be $500,000,000. For El Salvador, whose population in 1951 was approximately 2,250,000, Caceres Bustamante(5) reckoned the annual cost of malaria to the country at US $6,400,000, or approximately $2.85 per person.

Malaria control activities have been carried out in all republics of the Americas in which the disease is, or was endemic. Initially, such control was usually limited to urban centers. In these activities, engineers have played important roles. Prior to the discovery and extensive application of the chlorinated-hydrocarbon insecticides and others, whose residual properties are now well recognized, the engineer was often charged with planning and executing the control works and activities, which has for their purpose to so change the environment as to reduce the numbers of the vector species of Anopheles. The engineer was a member of a team of individuals, drawn from a variety of disciplines, working together to control a single disease. Thus, to simplify the concept, the physician studied the incidence of malaria in the population at risk, and its seasonal and annual variations, to obtain a knowledge of the disease in an area and a basis for the establishment of priorities in control. The biologist studied the habits of the vector or vectors, and indicated the kinds of alterations in the environment which were likely to reduce these in numbers. With this information the engineer could plan and design the necessary works, and choose the most practicable control means, both from the standpoint of economics and of effectiveness.

Parenthetically, it should be mentioned that in public works projects in malarious areas the danger always exists that the disease may be increased, if due regard is not taken of the effect of the proposed changes in environment on the production potential of the vector.

Malaria control, prior to World War II, was confined, in the majority of instances, to urban centers in which the concentration of population permitted a low per-capita assessment of control costs and in which financing of malaria control activities was possible.

With the advent of DDT and other insecticides of prolonged residual action against mosquitoes, it became possible for programs of control to be expanded away from urban areas, and to provide protection for the rural population as well. In some specific instances, engineering works continued to be more economical, but in the majority of countries in the Americas programs for the residual spraying of houses were organized and carried out. Thus, in the United States, the number of houses sprayed during 1945 was approximately 280,000, a figure which increased to 1,365,950 in 1948(6). In Venezuela, sprayings increased from 28,905 in 1946, to 874,850 in 1953(7).

Other countries report similar increases in the number of houses sprayed.

In these control programs based upon residual house spraying, the engineer continued to collaborate as part of the control team mentioned previously. Now, however, control activities had been placed in one basket, so to speak, and reliance was placed upon the single method of application of insecticides. In some instances, engineers who had previously worked in malaria control preferred to leave this particular field, and devote themselves to other activities. In other instances, engineers recognized a particular challenge in the choice of a single method of control and devoted their efforts to a reduction in unit cost of this method, so that protection could be afforded to the maximum number of people under fixed budgetary conditions.

Control programs, utilizing DDT and similar insecticides, are usually planned and evaluated upon the bases of incidence of the disease and per capita costs. Areas of high incidence receive priority, as do easily accessible areas in which it is possible to keep unit costs low. Spleen and parasite indices and data on morbidity and mortality from malaria, permit a comparison of the previous situation with that obtaining after several years of control, and an evaluation in terms of funds invested against fewer cases, fewer deaths, and increased production. Control programs, however, are continuing programs, in the sense that if stopped there exists the danger of invasion of malaria from untreated areas, with a return to the previous high incidence.

The extraordinary success of a number of these control programs has demonstrated that malaria can be eradicated from a previously endemic area through careful and persistent spraying of houses. Eradication, as defined by the National Malaria Society of the U.S., is considered to have occurred when three consecutive years have passed without the appearance of a single new local case (unless contracted from an imported case).

The concept of eradication, and the present urgency of its adoption, are founded upon three basic considerations[8]. First, it has been shown that should malaria transmission cease completely for a period of years, there is a spontaneous disappearance of the infections in the human population. Although some rare infections may live on or persist, should several years pass without transmission they will no longer be infective for mosquitoes. If, for example, in the case of Plasmodium vivax, a period of three years occurs without transmission, the cycle in the mosquito will not be completed, and the mosquito will be unable to infect man. For P. falciparum, a period of two years is considered sufficient.

The key phrase, then, in the eradication concept, is "complete interruption of transmission for a period of at least three years". You will note that malaria eradication does not imply eradication of mosquitoes. It does imply the killing of all infected mosquitoes, so that transmission cannot occur. It requires that all areas in which malaria is present be included within the eradication program, regardless of difficulties of access, or increased cost of treatment. It requires, finally, perfection in operations, a perfection not exacted of control programs, and seldom achieved by them.

Within the Americas, the application of this concept has eradicated malaria from Chile and from French Guiana, from a considerable part of Argentina, and from a very large area of Venezuela, under the definition as established by the National Malaria Society. Malaria is considered close to eradication from the U.S. and Puerto Rico, and from the island of Tobago. It has been eradicated from the coastal region of British Guiana.

A second consideration, leading to the urgency of eradication, is the appearance of resistance to chlorinated-hydrocarbon insecticides in some species of mosquitoes. This resistance may take the form of a change of behavior by the mosquito, as in the case of A. albimanus in Panama, which no longer enters sprayed houses in the same numbers as previously, nor are its resting habits the same[9,10]. Other malaria vectors such as A. sacharovi in Greece and the Near East, are resistant to the chemical itself, or can modify it in much a way that it is no longer toxic. A. sacharovi now exhibits resistance to DDT, Gammexane and Chlordane[11].

Should resistance by mosquitoes to DDT and similar insecticides become generalized, we shall have lost our most potent weapons in the fight against malaria. Unless new techniques could be developed, programs would again be reduced to those limited urban areas able to afford relatively high per capita costs of control, and the rural population would again be exposed to its number one health enemy. Hence the necessity to eradicate malaria before mosquitoes develop resistance to insecticides.

Another kind of resistance, also reported to be on the increase, leads to the third fundamental of eradication. This is the resistance exhibited by Ministers of Finance each year around budget time, when asked to continue supporting programs of malaria control. Governments, which see annual reports of malaria services stating that incidence has been reduced 85% or 90% from 1954 levels, are understandably reluctant to continue support for large scale control year after year, and are anxious to reduce operations so as to utilize savings in other activities. This would, of course, lead to a gradual invasion by malaria from untreated areas, and would ultimately require Governments to re-establish control programs at high levels of expenditure.

The economic advantage of the eradication concept over that of control lies in terms of the limited period required for eradication. Control programs may continue for years, but eradication has a fixed time limit. Operations will be required at high levels for four years, in order to achieve complete interruption of transmission for a three-year period, and should then be sharply reduced to the minimum level required to guard against imported cases. Eradication may thus be considered in terms of a short-time investment of funds for future dividends, dividends which accrue in terms of longer life, greater productivity, and increased economic potential.

The sum of these three important factors: spontaneous disappearance of infective forms of the parasite upon complete interruption of transmission; possible extension of already developed resistance by some anophelines to other species; and increased reluctance on the part of Governments to the perpetuation of control programs, has produced the present concept or eradication. As previously mentioned, the eradication program in the Americas was approved by the Pan American Sanitary Conference in October, 1954. The VIII World Health Assembly, meeting in Mexico City in May, 1955, approved a resolution establishing the eradication of malaria as a world-wide goal, and authorizing the World Health Organization to provide the necessary technical collaboration and coordination.

The coordination of these programs, and the establishment of uniform standards of operation and evaluation, is important as these activities develop on a hemisphere-wide and eventually world-wide basis. As malaria is eradicated from one country, it is important that neighboring countries be actively engaged in eradication as well, so as to reduce the danger of imported cases. For these purposes, the Pan American Sanitary Bureau has established in

Mexico City a "Coordinating Office of the Malaria Eradication Programs", which is charged with drawing up technical standards, and giving technical advice on the organization, operation and evaluation of such programs.

Malaria eradication programs are now in operation in Mexico, Haiti, the Dominican Republic, El Salvador, Venezuela, Argentina and in a number of states in Brazil. Existing control programs in other Central American countries, in Panama, Ecuador, and in a number of the Carribbean islands, are now being converted to eradication. It is expected that by mid-1957 all malarious areas in the Americas will be subjected to an all-out attack, with eradication as the goal.

In order to give an idea of the magnitude of the problem and of the requirements in terms of supplies, man-power and human ingenuity, the program planned by Mexico will be described briefly. The data presented are calculated for the year 1958, the mid-year of the program(12).

The malarious area of Mexico is calculated to include 1,477,000 $km.^2$, or 570,278 square miles. All but two of the 29 states of Mexico are affected by malaria, as are the Federal District and the territories of Quintana Roo and Baja California Sur. The population at risk is calculated at 16,746,136 persons, and the total number of sprayings of individual houses during 1958 is estimated at 4,135,712.

This amount of work will require 1,654 men working full-time at spraying, with additional supervisory personnel necessary, since to insure total coverage good supervision is essential. Insecticide will be consumed at the rate of approximately 15,000 lbs. daily, over the entire area, with corresponding problems of maintaining a continuous supply to personnel in areas where communications will be difficult.

In addition to the compression sprayers required, which must be maintained in usable conditions at all times, a total of 571 vehicles of various types will be used for transport of personnel and supplies. Maintenance of these vehicles will be of primary concern, since the eradication concept does not excuse the omission of houses because breakdowns in transportation did not permit them to be sprayed. Some 30,000 of the total sprayings will be done by squads operating in small launches and canoes and some 225,000 sprayings will be done by mounted squads.

For purposes of evaluation of the results of house spraying, some 487 technicians will be employed. Among their responsibilities will be the search for cases of malaria, searches for adult anophelines in houses, and vigilance for any signs of resistance by anophelines. Their number will be augmented by the necessary laboratory and specialized personnel.

One might now consider the engineering responsibilities involved in the Mexican campaign. All activities relating to spraying operations have been assigned to engineers. The country has been divided, for administrative, convenience, into 14 Zones, each responsible for roughly 300,000 sprayings annually. To each Zone will be assigned one or more engineers.

The first responsibility of the Zone Engineer is the training of the auxiliary supervisory personnel who will work under his direction. Although training courses for all classes of personnel have been established for some time in Mexico, the Zone Engineer must place the finishing touches required. At a later date, he will be responsible for the training of spraymen as well.

A second duty is the planning of the spraying operations per se within the Zone. For this planning, a geographic reconnaisance is required, so that all dwellings to be sprayed can be located and numbered to insure that none will

be missed. A study of the communications system within the Zone must be made, so that itineraries of spray squads may be planned properly. Due regard must be paid to differences between wet and dry-season conditions, since some localities may be isolated during the rainy period. Sub-bases of supply must be selected. Effective planning, prior to the initiation of spraying operations, will greatly assist in insuring that all houses in the malaricus zone which require spraying will be covered within the time limits established, and, equally important, within the budget for the program.

In certain situations it may not be necessary to spray all houses in the malarious area. In large population centers, it may be equally effective to spray only the peripheral houses, or to establish a barrier zone along one edge of the city. To return again to the concept of teamwork in malaria, such situations will be studied by the physician, the entomologist and the engineer, before a decision is taken regarding such measures.

Once spraying has begun, the engineer is responsible for seeing that 100% coverage is achieved. He must keep field personnel supplied with insecticides, and with equipment in good order. He must maintain the transportation system within the Zone, and be sure that preventive maintenance procedures for vehicles are carried out, so that economies in operation may be achieved. Above all, he must constantly supervise his subordinate personnel, so as to insure complete coverage.

The periodic evaluation by the engineer of costs of operation is essential if an economical program is to be carried out. The analysis of varying unit costs for different areas will often guide the engineer to the discovery of weaknesses which must be corrected, or to new techniques of operation which may profitably be generalized.

The final evaluation of the engineer's work will consist, of course, in the number of cases of malaria found within his zone of operations. Thus the entire program will be judged not on the basis of reduction of indices of malaria as was done in the past, nor on the cost per capita, but on the presence or not of new cases.

The cost of the program in Mexico, for the preliminary period of training, reconnaisance, and planning, and for the four years of intensive spraying, which will begin in September of this year, has been calculated at $250,000,000 pesos, of US $20,000,000. Of this amount, some US $12,000,000 will be provided by the Government of Mexico. The remaining US $8,000,000, in terms of supplies and equipment, is provided by the United Nations, through its Children's Fund (UNICEF).

To provide an idea of the magnitude of the malaria problem in the Americas, and of its challenge to health workers and Governments, Table I is presented to show, by countries and political divisions, the population now living in malarious areas. These data are taken from the latest figures furnished by countries to the Pan American Sanitary Bureau(13).

There is another field in which engineers can make important contributions to the malaria eradication programs. This is in the field of equipment, principally to the equipment used for the application of insecticides.

The compression sprayer has received wide use in programs for the spraying of houses. This type of sprayer was originally designed and produced to meet the requirements of agriculture. The requirements for malaria control are more severe than for agriculture, particularly in terms of the work required from a sprayer, and the punishment given to it. There are always upper and lower limiting factors in the design of a sprayer. It must be

TABLE I

POPULATION EXPOSED TO MALARIA IN THE AMERICAS (13)

Country or Territory	Population Presently Living in Malarious Areas	Observation
Argentina	1,655,000	Malaria eradicated from area of 200,000 population
Bolivia	600,000	
Brazil	30,890,000	
Canada	- 0 -	Not malarious
Chile	- 0 -	Malaria declared eradicated
Columbia	6,999,000	
Costa Rica	406,000	
Cuba	600,000	
Dominican Republic	1,071,000	
Ecuador	1,450,000	
El Salvador	1,282,000	
Guatemala	1,360,000	
Haiti	2,220,000	
Honduras	1,203,000	
Mexico	16,746,000	
Nicaragua	1,125,000	
Panama	500,000	
Paraguay	423,000	
Peru	3,773,000	
U.S. & Puerto Rico	- 0 -	Malaria close to eradication
Uruguay	- 0 -	Not malarious
Venezuela	1,558,000	Malaria eradicated from area of 2,565,000 population
Bahamas	- 0 -	Not malarious
Barbados	- 0 -	Not malarious
Bermuda	- 0 -	Not malarious
British Guiana	23,000	Malaria eradicated from area of 442,000 population
British Honduras	61,500	
French Guiana	- 0 -	Malaria eradicated
Guadaloupe	215,000	
Jamaica	209,000	
Leeward Is.	145,000	St. Kitts and Nevis are not malarious
Martinique	200,000	
Surinam	165,000	
Virgin Is.	- 0 -	Not malarious
Windward Is.	245,000	St. Vincent is not malarious
Trinidad	628,000	Malaria is close to eradication from Tobago
TOTAL	75,752,500	

durable, yet light enough so its weight will not tire a small man unduly. It must have certain essential operating parts and safety devices incorporated in the unit, and yet lend itself to rapid repair by unskilled personnel. The happy medium is not always easy to achieve.

The Expert Committee on Insecticides of the World Health Organization prepared specifications for compression sprayers in 1950. This Committee is drawn from a world-wide panel of engineers and other professionals who are actively engaged in insecticidal programs, whether for agriculture or health. The specifications have recently been revised by the Committee on the basis of experiences and research during the past five years, and represent the concensus of a large group of field and research workers. These specifications are used as guides by procurement agencies for malaria programs, and through collaboration with manufacturers it has been possible to produce superior equipment.

Two of a number of components of the compression sprayer which require further improvements might be mentioned. Most units are now operated in the 50 psi - 30 psi range, since the nozzles used are rated on their discharge and pattern at 40 psi. For one particular nozzle which is used by a number of programs, this results in a discharge which varies by 25% over the pressure range. The sprayman must, therefore, vary his rhythm of spraying as pressures decrease, which is difficult for him to do, or run the risk of over-treating (anti-economical) or under-treating, which reduces the period of residual activity, and may be a factor in the development of resistance by mosquitoes, through their exposure to sub-lethal doses.

A pressure regulator valve, which would provide a constant discharge at the nozzle, and which is operable under a variety of field conditions, would solve this problem. Although such valves are available, field experience with them has not been satisfactory so far. Improvements in design are required before such valves can be adopted for all programs.

The nozzle itself, as used in malaria programs, presents a design problem. If a nozzle were designed and produced which would operate effectively at, say, 10 psi, two important gains would be achieved. First of all, if used with an effective pressure regulator valve, it would permit the discharge of the contents of the sprayer with only two cycles of pumping by the operator, instead of the ten or twelve now required for work in the 50-30 psi range. The savings in work by the 1,654 spraymen in Mexico every day would be great indeed. Secondly, as a result of the lower operating pressure, such a nozzle would reduce atomization at the point of departure, and rebound of droplets from sprayed surfaces. This would mean less waste of insecticide on the one hand, and less danger of contamination to the operator on the other. The latter point becomes increasingly important as we begin to use, more and more, insecticides more toxic than DDT.

A final class of equipment which may be mentioned here as requiring improvement is that provided for the protection of personnel. Although not required for work with DDT, the more toxic insecticides oblige one to provide protection for the spraymen, so as to prevent inhalation of the insecticide, or absorption through the skin. Protective equipment currently available is not entirely suitable, particularly under tropical conditions, and further developments are necessary.

To summarize, the Malaria Eradication Programs now in operation in the Americas, or shortly to be initiated, represent the largest coordinated attack on a single disease yet undertaken in the Hemisphere. The need and urgency

for their establishment are well recognized. They are bold programs, in the sense that the eradication concept admits only two possible end results: complete success, or failure. The mere reduction in incidence of malaria is not enough; it must be totally eliminated, and if only a few residual cases remain, eradication will not have been achieved.

These programs thus require a perfection in planning, in operations and in evaluation, and it would seem that their principal challenge to engineers consists precisely in this grade of perfection exacted. In addition to the high technical capacity demanded of engineers, Ganan[14], Engineering Director of the Eradication Program in the Dominican Republic, has perhaps best expressed the additional requisites expected, when he says, "Eradication requires something more than equipment, supplies or techniques. It demands that its personnel possess affection for their work, and particular interest in achieving the stated goal".

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INVESTIGATIONS OF THE HYDROLOGY OF SMALL WATERSHEDS IN TEXAS[a]

Trigg Twichell,[b] A.M. ASCE
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INTRODUCTION

A comprehensive knowledge of the Nation's water resources and a continuous inventory of these resources are fundamental to the economy of the community, the State, and the Nation.

Man, through his efforts to sustain and improve the economy of a growing population with the passing of time, increases the demands upon the soil, minerals, plants, and water resources of the Nation. The varying manner in which these resources are developed and used affects the quantity and quality of the water resources. It is important, therefore, that basic data be obtained on a continuous basis to evaluate (1) the natural occurrence, quantity, and quality of water for a region or river basin through extended climatic cycles, (2) the interrelationships of surface and ground water, and (3) the effects of man's developments upon the water resources. Increased accuracy of basic data on these occurrences is required as water demands approach or equal nature's water supply to the affected area.

In 1888, the U. S. Geological Survey began investigations of the water resources of the Nation, and the first gaging station in Texas was established on the Rio Grande at El Paso, May 10, 1889. Since 1915, the Geological Survey, in cooperation with the Texas Board of Water Engineers, the U. S. Corps of Engineers, the U. S. Soil Conservation Service, U. S. Bureau of Reclamation, and municipal and other State agencies, has conducted basis hydrologic investigations in Texas for evaluating the water resources. The earlier work was confined to larger streams. With increases in funds, investigations were broadened to include river tributaries, ground water, chemical quality and sediment studies. As water development progressed, investigations have expanded when possible to provide data for use in determining the effects of development upon natural sources of supply.

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a. Presented at the National Meeting, ASCE, Dallas, Tex., February 15, 1956.

b. Dist. Engr., Surface Water Branch, Texas Dist., Geological Survey, U. S. Dept. of the Interior, Austin, Tex.

Today's hydrologic investigations lag development. The major deficiency is the lack of basic hydrologic data for evaluating the natural occurrence of water resources for small streams. The effects of water-control and water-use projects on the water resources of streams having drainage areas between 1 and 100 square miles are understood only to a limited degree. Hydrologic investigations of small watersheds require the collection of proportionately more detailed information than is needed for the larger streams. Rainfall and runoff patterns must be observed on small watersheds to determine the variations resulting from irregular rainfall and other factors affecting runoff.

The cost of conducting such investigations is relatively high, but the work must be done if we are to evaluate the full effect of developments on the natural water resources. In Texas, water-use and water-conservation projects of all types are mushrooming at such a pace that it is difficult to appraise the relative effect of one development upon another. The development of Texas' water resources, therefore, has reached a critical stage. Projects that have been designed to utilize all or a given portion of the flow of a stream at a given location are likely to operate below design capacity in the future because some water originally available to the project may be used by other upstream developments. Likewise, projects designed to use only the natural base flow - that furnished to the stream by ground-water reservoirs in the form of springs and seeps - may be forced to operate below design capacity because the natural base flow is reduced by ground-water diversions.

Historical streamflow records that have been used in designing existing river developments can no longer be used for determining future water yields without first considering possible effects of upriver water-use, water-control, and water-conservation measures. An analysis of the various water developments reveals that downriver supplies may be increased under some operations and reduced under others. For example, the long periods of no flow that were experienced on the Trinity River at Dallas during the 5-year drought of 1909-1913 will never occur again unless industrial waste and sewage effluent are consumed in the Fort Worth-Dallas metropolitan area. Prospects are that this source of supply from sewage effluent will increase low river flows for years to come, even though facilities may be provided to re-use such water sources. In contrast to this increase in low flows in the upper Trinity River, farmers and others who have used the former dependable flows of Comanche Springs (average discharge 42.7 cfs) at Fort Stockton in West Texas, and Comal Springs (average discharge 309 cfs, Texas' largest) at New Braunfels in central Texas are now faced with widely fluctuating spring flows as a result of a large ground-water withdrawals from the common underground reservoirs of these separated regions. Owing to heavy ground-water pumping the Comanche spring flow at times is reduced to zero during the irrigation season, and the flow of Comal Springs dropped to an all time low of 41 cfs Aug. 30, 1955 because of municipal, industrial and irrigation demands during the summer months, and reduced recharge from rainfall and low-stream flow. These are extreme conditions but they illustrate the competitive manner in which the water resources are being used.

It must be recognized that water use usually results in some water consumption. The consumptive rates vary with types of use, the variations of climate, and with the changes in hydrologic factors controlling water yields.

Historical streamflow records, with few exceptions, now serve only as a base from which current and future flow records may be evaluated. Dependable factors or coefficients for adjusting old runoff records to current or

future conditions will not be available until sufficient basic hydrologic data have been collected to show the practical effects of all forms of water development and conservation measures.

The following are the principal man-made factors that are believed to be changing the amount or distribution of streamflow received from a given amount of precipitation and the commonly accepted effect of these factors:

Man-made factors	Runoff	
	Increase	Decrease
1. Vegetal reduction, over grazing and timber cutting	X	
2. Strip cropping and rotation of crops		X
3. Terracing and contour plowing	a/X	a/X
4. Stock tanks		X
5. Water spreading on pastures		X
6. Irrigation		X
7. Industrial processing and cooling		X
8. Domestic uses		X
9. Water-flow retardation structures		X
10. Reservoirs impounding water for any use		X
11. Rainfall on reservoir water surface	X	
12. Reservoir seepage losses	b/X	b/X
13. Ground-water withdrawals		X
14. Highways, roads and urban areas	X	

a/ Result is dependent upon rainfall intensities and duration of storm
b/ Result dependent upon subsoil geology and whether seepage water again reaches stream or is consumed by beneficial or nonbeneficial plants.

Current Water Uses and Developments

Demands for surface water far exceed the low flow of every major Texas stream.

If it were possible to compare a 1941 aerial photograph of Texas with one made today we could readily see the marked contrast in soil and water conservation, water-use and water-control structures. An air view today would show improved soil and soil-moisture conservation practices in every section of the State. More than 150,000 stock tanks and farm and recreation ponds are operating in the State, and the number of 200-acre-foot conservation reservoirs is increasing at a steady rate. Twenty-two major multiple-use reservoirs, such as Benbrook, Garza-Little Elm, Hords Creek, Lake Colorado City, Lake Houston and others, each having a capacity of 5,000 acre-feet or more, have been constructed in the past 10 years. Data are not available to evaluate the number of the smaller structures or the quantity of water beneficially or otherwise used or retained by small reservoirs and stock tanks, but the total quantity of water controlled by these operations may be a significant factor in water supply in drought or arid areas.

Records of the State Board of Water Engineers show that 79 large reservoirs, each having a capacity of 5,000 acre-feet or more, have been constructed in Texas for municipal, industrial, irrigation, hydroelectric and flood control uses prior to 1956. The combined capacity of these reservoirs at spillway crest is 25,300,000 acre-feet, of which 12,100,000 acre-feet is classified as flood storage, 1,600,000 acre-feet as dead storage, and 11,600,000 acre-feet for municipal, industrial, irrigation and hydroelectric uses. These

reservoirs are distributed through the major river basins as follows:

Basin	Number
Canadian	1
Red	12
Sabine	1
Neches	2
Trinity	14
San Jacinto	2
Coastal	4
Brazos	18
Colorado	13
Guadalupe-San Antonio	5
Nueces	2
Rio Grande-Pecos (Texas)	6
Total	80

With the above water-use facilities it is possible to impound large quantities of water in every major river basin in the State. This fact is demonstrated more frequently each passing year. During a 24-hour period Sept. 9-10, 1952, 699,000 acre-feet of water was stored in Lake Travis on the Colorado River above Austin and more than 1,000,000 acre-feet was stored in Buchanan Reservoir and Lake Travis or the total runoff from that storm in the Colorado River basin. Geological Survey Water-Supply Paper 1260-A, "Floods of September 1952 in the Colorado and Guadalupe River Basins, Central Texas", by S. D. Breeding and J. H. Montgomery, contains the following statements on pages 1 and 45:

> "Following a severe drought floods of exceptional size occurred in the central Texas 'hill country' as a result of heavy rains during the period Sept. 9-11, 1952. As much as 26 inches of rain fell in the Guadalupe River basin and in the central and lower Colorado River basin. x x x
>
> "Record floods occurred on many large and small streams. Inflow into Lake Travis on the Colorado River reached 803,000 cfs from a 6,650 square mile contributing area.
>
> "Lake Travis effectively controlled the flow of the Colorado River below the lake."

A flood of even greater magnitude occurred in the central Rio Grande basin late in June 1954. About 2,000,000 acre-feet of runoff from that record-breaking flood was impounded in Falcon Reservoir and was later used for irrigation in the lower valley.

Nature has its own way of retaining moisture in the soil and feeding underground reservoirs. The storm of Sept. 24, 1955, on the upper Nueces River produced a peak discharge of 310,000 cfs at the Laguna gaging station with a total flood runoff at that point of only 143,000 acre-feet. As this flood combined with the major flood from the West Nueces River and moved across the Balcones fault area, 50,000 to 70,000 acre-feet percolated to underlying gravels and limestones. Further losses occurred downriver to the Carizzo sands and other ground-water aquifers. When the flood reached Three Rivers, above Lake Corpus Christi, the peak discharge had fallen to 3,360 cfs and the total upriver flood volume had dropped to 42,700 acre-feet.

Hydrologic Data on Small Watersheds

The successful operation of existing projects and the proper development of new projects is contingent upon basic hydrologic data from which groups of small developments, and each major development within a large watershed, may be operated in a coordinated manner with one another.

In recent years, the farmer and the rancher have recognized the value and convenience of water conservation and surface-storage practices on the land. The application of these practices now extends in varying degrees to every section of the State. These conservation and water-use measures naturally consume water in varying amounts, and today there is a wide difference of opinion as to what extent surface-water supplies are or are not being reduced and the extent to which ground-water reservoirs are or are not being recharged by this expanding land-management operation.

In order to obtain basic data for use in evaluating the effects of land-management practices and flood-detention structures on runoff, the Geological Survey has in recent years collected streamflow data for this purpose in various sections of the United States. The purpose of the investigations is to collect basic data for use in determining rainfall-runoff relations, and the effects of soil- and water-conservation practices and water-flow retardation structures on runoff in watersheds of less than 100 square miles. Such investigations began in Texas in 1951 and are conducted in cooperation with the Texas Board of Water Engineers and the U. S. Conservation Service.

The Texas Board of Water Engineers and the cities of Fort Worth and Dallas have requested that similar basic data be obtained on four small watersheds in the Trinity River basin above Dallas. The purpose of these studies is to collect basic data for use in determining the effect of small-stream development upon the surface-water resources in that basin. This investigation will be financed by the Geological Survey, the two cities, and the State.

Stock Tanks

Various types of hydrologic research are being conducted by the Geological Survey, one of which covers stock tank operations in arid areas. A review of the findings is of much interest to those concerned with water development.

Geological Survey Circular 110, entitled "Hydrology of Stock-Water Reservoirs in Arizona", by W. B. Langbein, C. H. Haines and R. C. Culler, in a discussion on design, page 13, states:

> "The problem is to determine how much of an observed recession in water level during dry weather periods is due to seepage and how much is due to evaporation. The principles employed in the separation are: (1) Evaporation varies in response to meteorologic controls, and (2) seepage is relatively uniform at a given stage and season. The relative proportions of evaporation and seepage are therefore variable.
>
> "In the annual cycle, loss by evaporation from shallow lakes reaches maximum in summer and minimum in winter. x x The rate of recession ranged from a maximum of 0.98 foot per month in June and July to a minimum of 0.26 foot per month in January. It is evident that evaporation is the dominant factor in this seasonal variation of water loss . . ."

Evaporation losses recorded in Arizona are of the same order of magnitude as losses recorded on large and small reservoirs in West Texas. Therefore, it is likely that the principles of small reservoir design should, in general, be similar to those in Arizona.

Regarding the principles of reservoir design, the Arizona report states on page 17.

> "Unlike an irrigation reservoir, the performance of a stock-water reservoir is dependent on depth of water rather than on capacity. The records demonstrate that there is generally little need for a reservoir to have a capacity greater than that necessary to store the mean annual runoff. Providing additional capacity to store the water that would spill in years of extraordinary runoff, according to the evidence obtained, does not thereby provide water during extended dry periods. Rates of loss are great at high infrequent stages and losses at such stages may be at the expense of downstream users without necessarily benefitting the stockwater supply. x x x"

The studies found that to provide stock water in Arizona for reasonably long dry periods the depth should be at least equal to $15(2/(F+1))L$, where L is the average rate of water loss in feet per month and F is the average frequency of recharge per year, provided that the capacity at this depth does not greatly exceed annual runoff.

Hydrologic data - small areas in Texas

Although the special watershed investigations in Texas began with initiation of work on Honey Creek near McKinney, Texas, in 1951, full instrumentation for hydrologic studies and reservoir construction was not sufficiently advanced to obtain many basic data until 1954. Through this 3-year period, similar investigations were started on seven other watersheds. These are so located that most factors affecting runoff will have considerably different effect in one watershed than in any of the other watersheds. Surface geology, soils, vegetation, evaporation, transpiration, and rainfall are some factors that may vary from watershed to watershed.

It is believed that the data showing rainfall and resulting runoff, variations in runoff, water losses recorded in integral parts of each watershed, inflow, outflow, and water losses from selected reservoirs, and the recorded net runoff from the entire watershed through years of climatic variations from existing projects. It is believed also that the data will be useful to improve the design and proper operations of future soil and moisture and waterflow-retardation facilities.

All of the watersheds under study in Texas are considered by the U. S. Soil Conservation Service to be demonstration projects. That agency has surveyed each watershed and prepared a report on the following major subjects: (1) Description of watershed; (2) Economy of the watershed; (3) Flood problems and damages; (4) Flood control activities; (5) Land treatment activities; (6) Hydraulic and hydrologic investigations; (7) Sediment conditions; (8) Flood plain scour and channel enlargement; (9) Flood damages; (10) The remedial program and its evaluation; and (11) Annual maintenance.

Basic hydrologic data as to runoff for watersheds of this size are practically nonexistent for Texas streams.

The collection of basic hydrologic data on the projects and the correlation of such data with physical changes in the watershed will be valuable not only in the design of future structures, but also in determining water losses of the large streams to which such controlled streams contribute flow. Hydrologic investigations are now being conducted in the following watershed:

Watershed	Drainage Area Square Miles
Trinity Basin	
Honey Creek near McKinney	39.0
Brazos Basin	
Green Creek near Alexander	45.5
Cow Bayou near Mooreville	79.6
Colorado Basin	
Deep Creek near Mercury	43.9
Dry Prong Deep Creek near Mercury	8.31
Mukewater Creek near Trickham	70.4
San Antonio Basin	
Calaveras Creek near Elmendorf	81.0
Escondido Creek at Kenedy	82.2

Hydrologic instrumentation on the above projects was not completed in all watersheds until 1955. A number of flood detention reservoirs are yet to be built, but as soon as this is done, instruments will be installed and the investigations expanded to include these changes. The complete record of precipitation, amount and intensities, total runoff, water losses at the sites of the structures, therefore, begins with the 1955 water year.

The work is sufficiently established in the Deep Creek watershed near Brownwood and Honey Creek near McKinney to illustrate the types of data that are being obtained in sub-humid East and semi-arid West Texas.

Deep Creek

These data show also the effectiveness of the flood detention reservoirs in the Deep Creek watershed (See figure 1) for two major storms, one producing a rainfall of 7.39 inches in 12 hours (7.74 inches total for storm) Oct. 3-4, 1953, and the other a rainfall of 8.45 inches in 36 hours May 17-19, 1955.

The Deep Creek watershed is approximately 30 miles southwest of Brownwood where records of precipitation have been collected by the U. S. Weather Bureau since 1892. An analysis of these records for the 63-year period, 1892 to 1954, shows that the greatest 24-hour rainfall at that location was 5.28 inches Sept. 6-7, 1910, and that there was a total of twenty-eight 24-hour periods when the rainfall exceeded 3 inches. The following table gives the number of storms producing 24-hour rainfall as indicated:

Station Period	Average Annual Rainfall Inches	Number of storms producing 24-hour rainfall within the quantities shown			
		3"-4"	4"-5"	5"-6"	6"-7"
Brownwood, 1892-1954	27.44	18	7	3	0

Climatic conditions at Deep Creek are similar to those at Brownwood; therefore, the storms that occurred at Deep Creek Oct. 3-4, 1953 and May 17-19, 1955 are of an unusual character and may not be expected to occur frequently in the future. It should be recognized, however, that storm rainfalls of more than 20 inches have occurred at a number of West Texas locations and such a storm may be expected to occur over Deep Creek at rare intervals.

Figures 2 and 3 show graphically the rainfall on the Dry Prong Deep Creek Reservoir watershed and Deep Creek Reservoir No. 3 watershed for 1954 and 1955 water years. Rains were mostly erratic and of an explosive nature. The rainfall for 1954 was considerably below the average and the 1955 rainfall was greater than the average annual precipitation.

Honey Creek

The watershed is 10 miles northwest of McKinney, where records of rainfall have been collected by the U. S. Weather Bureau since 1903. The following table gives the number of McKinney storms producing 24-hour rainfalls as indicated.

Station and Period	Average Annual Rainfall Inches	Number of storms producing 24-hour rainfall within the quantities shown				
		3"-4"	4"-5"	5"-6"	6"-8"	8"-
McKinney, 1903-1953	39.24	21	12	5	1	0

Honey Creek is in a rainfall belt comparable to McKinney and a region which has materially more rainfall and runoff-producing storms than occur at Deep Creek in West Texas. Rainfall records at Honey Creek for 1954 and 1955 show that no storm of unusual character occurred during those years. The greatest 24-hour rainfall was 3.98 inches Sept. 30-Oct. 1, 1954, and the total rainfall during that storm was 4.68 inches.

Figures 4 and 5 show graphically the 1954 and 1955 rainfall, respectively, for Honey Creek Tributary at Reservoir No. 12 and Honey Creek Tributary at Reservoir No. 11. Rainfall for these watersheds was at a fairly uniform rate and the yearly precipitation was close to the long-time average.

Typical watershed investigation

Deep Creek and Dry Prong Deep Creek, Figure 1, are typical of the hydrologic instrumentation and the flood-detention reservoirs installed or to be installed in each demonstration watershed. The combined drainage area of the two streams is 52.2 square miles. Forty-six percent of this area lies upstream from the six combination water-conservation and flood-detention reservoirs that are operating in these watersheds. The combined conservation and dead-storage capacity of the six reservoirs is 826 acre-feet, or 15.8 acre-feet per square mile of drainage area. The combined flood storage in the six reservoirs, in addition to the conservation and dead storage, is 6,392 acre-feet, or 122 acre-feet per square mile of drainage area. The reservoirs are planned for automatic control of flood flows resulting from storms of 25-year frequency. Floods resulting from lesser storms are temporarily retained in the detention pools with flood flows discharging through fixed outlets at the top of each conservation pool. Floods of a magnitude greater than 25-year frequency are expected to fill the reservoirs and the excess flows will discharge through the emergency spillways.

Hydrologic instrumentation consists of a streamflow station at the lower reach of Dry Prong Deep Creek and on the lower reach of Deep Creek. These stations provide a continuous record of integrated flow leaving each watershed area. This is the flow that originates between the gaging station and all upstream reservoirs plus flow passing through each reservoir.

A continuous record of reservoir level and content is maintained by stage recorders at the reservoir at Site 8 on Dry Prong and at the reservoir at

Site 3 on a headwater tributary of Deep Creek. Current-meter measurements have been made to define the discharge curve for the drop outlet at each reservoir. The drop outlet is the fixed opening through which flood pool water drains. Area and capacity curves for the reservoirs have been developed from topographic surveys. No spring water feeds these streams; consequently, inflow occurs only with rainfall. Reliable records of inflow and outflow, reservoir content and reservoir losses are readily obtained from the records collected.

Records of content of the other four reservoirs (Sites 1, 2, 4, and 5) are obtained from weekly staff-gage readings and capacity tables.

Thirteen non-recording rain gages and 2 continuous recording rain gages supply a good record of rainfall and its intensities.

Land management practices, including contour farming, cover cropping, terracing, brush control, pasture planting, stock and fish ponds, have been applied to about 25% of the Honey and Deep Creek watersheds.

The studies have not yet provided for determining the possible groundwater recharge, sediment loads carried by the stream, or channel losses below the watershed under investigation. It is hoped that these phases of investigation can be added at some future date.

Soils

Deep Creek watershed soils are described by the Soil Conservation Service in its preliminary report as follows:

> "The Edwards Plateau soils are found at the top of the steep escarpment that rings the west, south, and southeast part of the watershed. These fine texture soils cover about 25 percent of the watershed. Reddish prairie soils occupy the central portion, or 70 percent of the area in the watershed. These soils are 88 percent fine textured and 12 percent medium textured. The remaining 5 percent of the watershed area x x x is cross timbers soils. These soils are 59 percent medium and 41 percent coarse textured."

The U. S. Department of Agriculture "Soil Survey of Collin County" shows the Honey Creek watershed soils to be Houston Black Clay, Catalpa Clay, and Bell Clay. It reports the surface and sub-surface soils are readily penetrated by water.

A brief review of the 1954 and 1955 rainfall and runoff records collected in the Honey and Deep Creek watersheds illustrates the type of data being obtained and is suggestive of some ways in which the data may be applied to practical problems. It must be recognized that a two-year record of such data is not sufficient for drawing conclusions. Such data must extend through several years to be of substantial value in hydrologic analysis. The comparison of the 1954 and 1955 precipitation records with the longer records at nearby stations illustrates this fact quite well.

Runoff volumes

The 1954 and 1955 runoff pattern for the reservoir watersheds on Dry Prong Deep Creek and on Deep Creek tributary are similar (see figs. 2 and 3). Runoff, like rainfall, occurred mostly during intense storms. Precipitation for the area above Dry Prong Deep Creek reservoir in 1954 was 18.28 inches or nearly 7.0 inches below the long-time average annual rainfall; yet, the yearly runoff was equivalent to 3.65 inches of rainfall equally distributed over the 4.32 square miles of watershed. Of the annual runoff, 3.07 inches, of 83%, occurred during the storm of Oct. 3-4, 1953 with the remaining

runoff mostly occurring in two smaller one-day storms, April 30 and May 11.

Although rainfall and runoff were greater in 1955 than in 1954, the natural water losses were greater also, and as a result, the differences between rainfall and runoff in 1955 were greater than in 1954.

The 1954 and 1955 runoff pattern for watershed contributing to Reservoir Nos. 11 and 12 in Honey Creek, was quite different from that of the west Texas stream. (See figs. 4 and 5).

Rainfall was more evenly distributed through the year and ranged from 28.28 to 35.58 inches on the two watersheds. Runoff occurred for longer periods of time, but the annual yield was relatively much less than that for the Deep Creek basins of West Texas.

Factors which caused the low runoff rate are not readily identified. This will require additional years of records and more detailed analysis of the data to evaluate the runoff factors.

Floods

Storms which produced the unusual floods of Oct. 3-4, 1953 and May 17-19, 1955 on the Dry Prong Deep Creek Reservoir watershed have been discussed briefly under the "Deep Creek" section. Figure 6 shows inflow-outflow discharge hydrographs, curves of accumulative rainfall and accumulative runoff for each of these storms. These graphs demonstrate the effective manner in which the high flood flows were controlled. The 7.74 inches of rainfall on the night of Oct. 3-4, 1953 produced a peak inflow to the reservoir of 1,580 cfs. The outflow from the reservoir did not exceed 52 cfs. The peak inflow of 2,550 cfs May 17, 1955 was regulated in a similar manner.

Reservoir losses

The impounding of water in a reservoir introduces water losses to surface-water supplies. These losses may result from evaporation, transpiration, and seepage. In humid areas and heavy rainfall regions evaporation losses may be offset by rains that fall on the reservoir surface. Seepage losses may be reduced to a low quantity by locating the reservoir on impervious soils, and transpiration losses are low where plant and aquatic growth is small.

In the western half of Texas, however, rainfall is low and evaporation losses from open-water surfaces range from 4.0 to 10.0 feet in a dry year. This water loss therefore is a major problem. Those planning any type of reservoir ranging from stock tanks upward to the largest possible reservoir, in arid or semi-arid regions, should weigh carefully water losses versus water need.

Table 1 summarizes the 1955 inflow and outflow at the six flood-retarding and water-conservation reservoirs operating in the Deep Creek basin. Shown also is the water passing out of the Dry Prong Deep Creek and Deep Creek basins during the 1955 water year.

Of the 8,659 acre-feet of inflow to the six reservoirs, 6,812 acre-feet passed downstream as outflow, and 1,846 acre-feet, or 21.5% was held as storage in the six reservoirs. Of the 1,846 acre-feet of stored water, 76 acre-feet were used for irrigation, 1,197 acre-feet was lost to evaporation, transpiration and seepage, and 573 acre-feet remained in storage at the end of the year.

Water detained in the Dry Prong Deep Creek Reservoir amounted to 16.4 percent of the adjusted runoff at the lower streamflow station (drainage area, 8.31 square miles).

Water detained in the five Deep Creek reservoirs amounted to 7.0 percent

of the adjusted runoff at the lower streamflow station (drainage area, 43.9 square miles).

This brief analyses of hydrologic data collected over a two-year period does not justify conclusions. The primary purpose of this presentation is to illustrate the type of data that must be collected for a much longer period of time to determine the true effects of upstream development on runoff.

SUMMARY OF FLOOD RETARDING AND WATER CONSERVATION RESERVOIRS IN THE DEEP CREEK BASIN

Table 1 McCulloch County, Texas, for the year ending Sept. 30, 1955

	Drainage Area	Inflow (Runoff)	Outflow	1955 Runoff held in storage at end of year	Drawn from runoff stored prior to 1955	Net change in storage during year	Losses from Reservoir during year				Depletion of 1955 flow at reservoir (Inflow minus Outflow)		a/Flow at Gaging Station		
							Evaporation and Seepage		Irrigation				Observed	Adjusted for Reservoir Storage	Depletion at reservoirs in percent of adjusted runoff at downstream stations.
	Square Miles	Acre-feet	Acre-feet	Acre-feet	Acre-feet	Acre-feet	Acre-feet	% of inflow	Acre-feet	% of inflow	Acre-feet	% of inflow	Acre-feet	Acre-feet	
Dry Prong Deep Creek Reservoir #8	4.32	2,190	1,580	132	0	+132	402	18.4	76	3.5	610	27.9			
Dry Prong Deep Creek at gaging station near Mercury, Tex.	8.31												3,100	3,710	16.4
Deep Creek Reservoir #1	9.19	2,642	2.198	133	0	+133	311	11.8	0	-	444	16.8			
Deep Creek Tributary Reservoir #2	.79	305	236	25	0	+ 25	44	14.4	0	-	69	22.6			
Deep Creek Tributary Reservoir #3	3.42	1,380	956	143	0	+143	281	20.4	0	-	424	30.7			
Deep Creek Tributary Reservoir #4	1.22	372	334	6	0	+ 6	32	8.6	0	-	38	10.2			
Deep Creek Tributary Reservoir #5	5.27	1,769	1,508	134	0	+134	127	7.2	0	-	261	14.8			
Total for Reservoirs	19.89	6,468	5,232	441	0	+441	795	12.3	0	-	1,236	19.1			
Deep Creek at gaging station near Mercury, Tex.	43.9												16,300	17,536	7.0

a/ Gaging station is below flood retarding reservoirs and measures residual runoff of basin above that point.

DEEP CREEK BASIN RESERVOIRS IN McCULLOCH COUNTY, TEX.

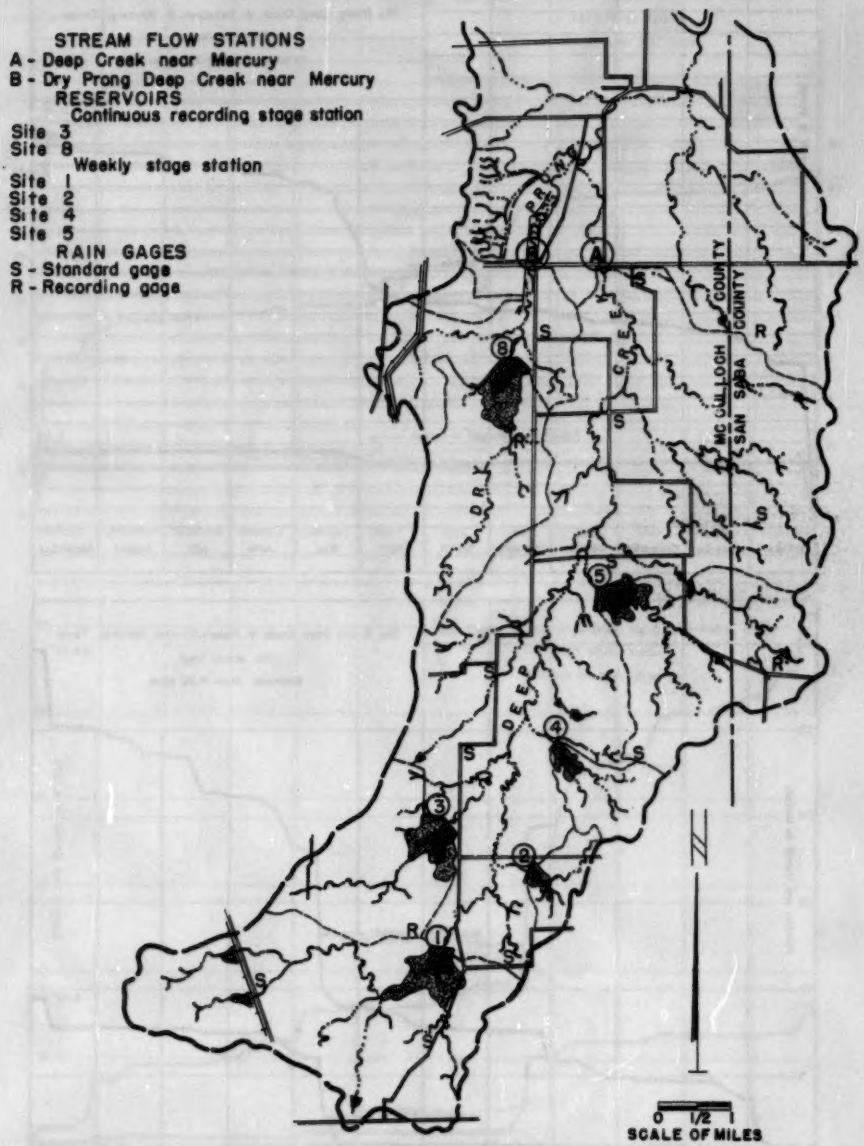
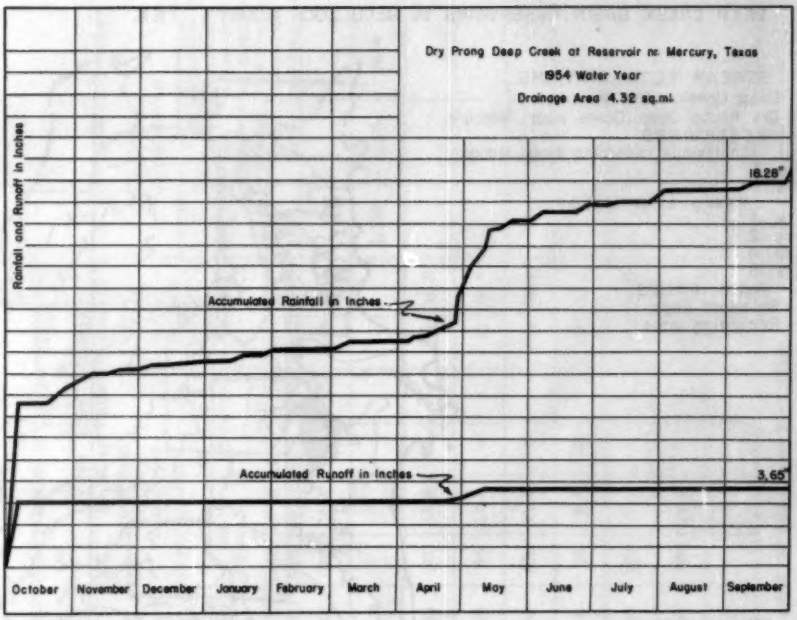


Fig. 1


Dry Prong Deep Creek at Reservoir nr. Mercury, Texas
1954 Water Year
Drainage Area 4.32 sq.mi.
Rainfall and Runoff in Inches
0
2
4
6
8
10
12
14
16
18
Accumulated Rainfall in Inches
Accumulated Runoff in Inches
16.28"
3.65"
October
November
December
January
February
March
April
May
June
July
August
September

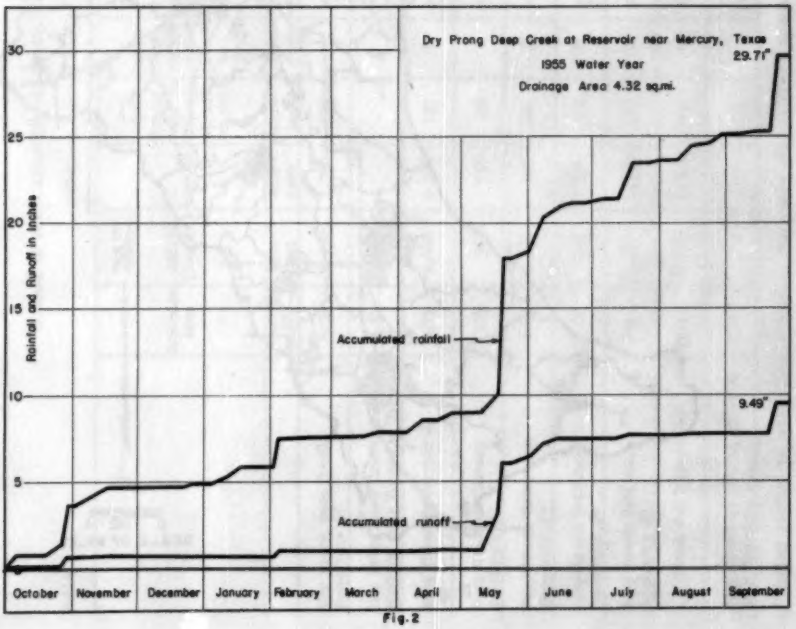

Dry Prong Deep Creek at Reservoir near Mercury, Texas
1955 Water Year
Drainage Area 4.32 sq.mi.
29.71"
Rainfall and Runoff in Inches
5
10
15
20
25
30
Accumulated rainfall
Accumulated runoff
9.49"
October
November
December
January
February
March
April
May
June
July
August
September

Fig. 2

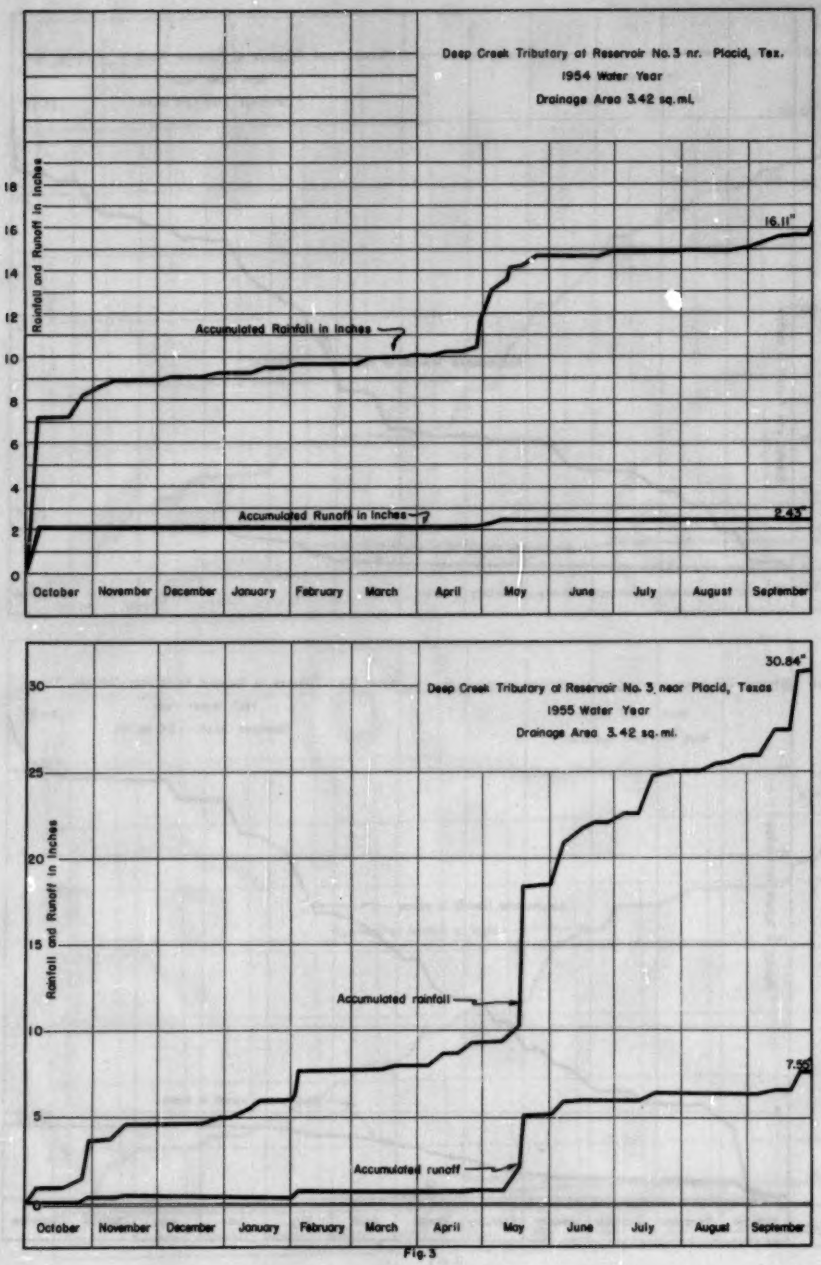
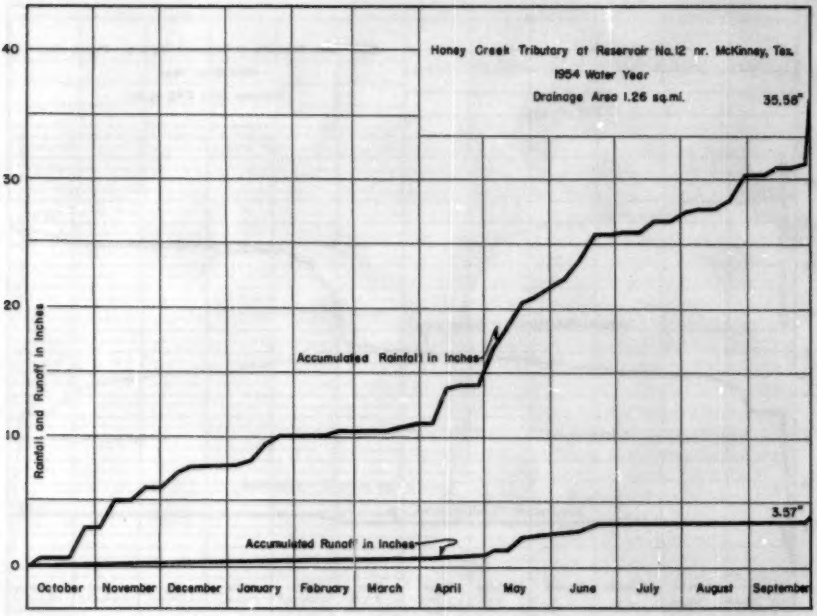

Deep Creek Tributary at Reservoir No. 3 nr. Placid, Tex.
1954 Water Year
Drainage Area 3.42 sq. mi.
Rainfall and Runoff in Inches
Accumulated Rainfall in Inches
16.11"
Accumulated Runoff in Inches
2.43"
0
2
4
6
8
10
12
14
16
18
October
November
December
January
February
March
April
May
June
July
August
September
Deep Creek Tributary at Reservoir No. 3 near Placid, Texas
1955 Water Year
Drainage Area 3.42 sq. mi.
30.84"
Rainfall and Runoff in Inches
Accumulated rainfall
Accumulated runoff
7.55
0
5
10
15
20
25
30
October
November
December
January
February
March
April
May
June
July
August
September

Fig. 3


Honey Creek Tributary at Reservoir No.12 nr. McKinney, Tex.
1954 Water Year
Drainage Area 1.26 sq.mi.
35.56"
Accumulated Rainfall in Inches
Accumulated Runoff in Inches
3.57"
Rainfall and Runoff in Inches
0
10
20
30
40
October
November
December
January
February
March
April
May
June
July
August
September

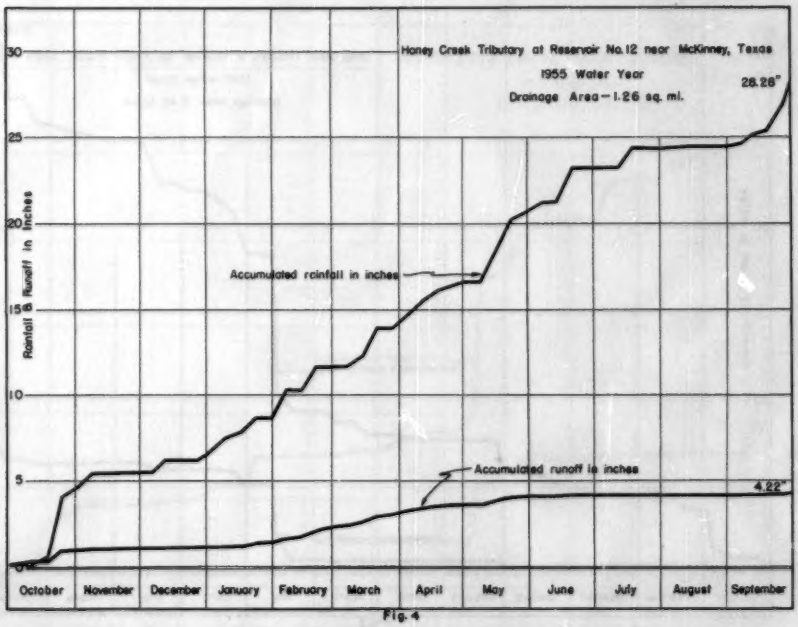

Honey Creek Tributary at Reservoir No.12 near McKinney, Texas
1955 Water Year
Drainage Area—1.26 sq. mi.
28.28"
Accumulated rainfall in inches
Accumulated runoff in inches
4.22"
Rainfall & Runoff in Inches
0
5
10
15
20
25
30
October
November
December
January
February
March
April
May
June
July
August
September

Fig. 4

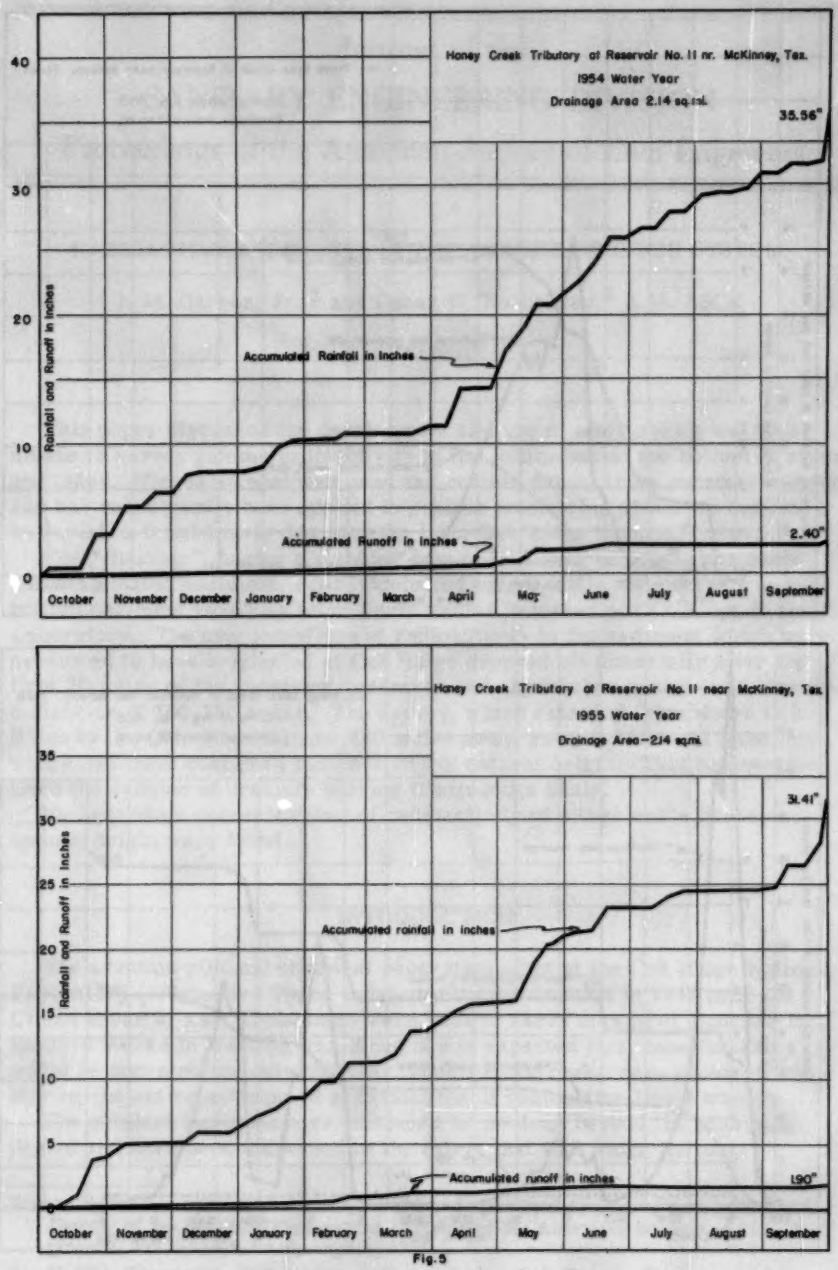
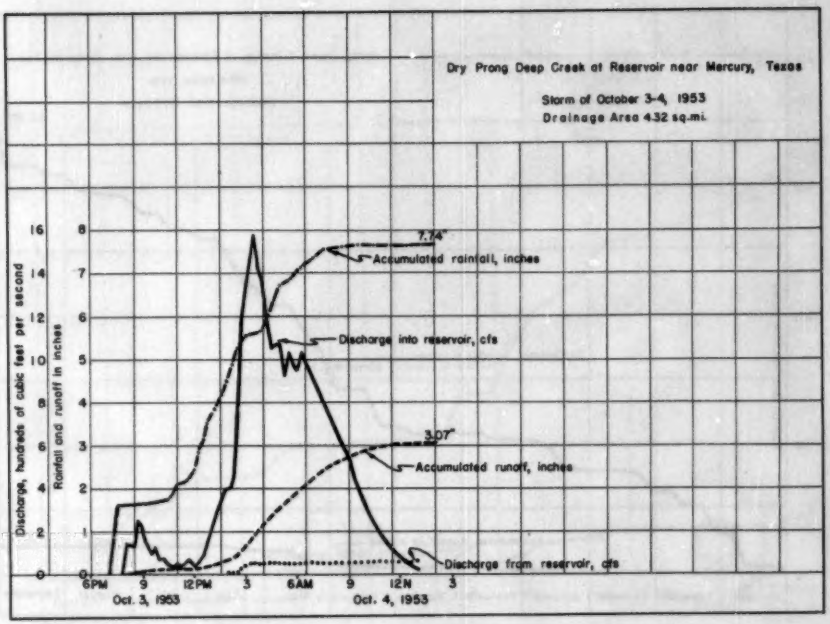

Honey Creek Tributary at Reservoir No. II nr. McKinney, Tex.
1954 Water Year
Drainage Area 2.14 sq.mi.
35.56"
Accumulated Rainfall in Inches
Accumulated Runoff in Inches
2.40"
Rainfall and Runoff in Inches
0
10
20
30
40
October
November
December
January
February
March
April
May
June
July
August
September
Honey Creek Tributary at Reservoir No. II near McKinney, Tex.
1955 Water Year
Drainage Area—2.14 sq.mi.
31.41"
Accumulated rainfall in inches
Accumulated runoff in inches
1.90"
Rainfall and Runoff in Inches
0
5
10
15
20
25
30
35
October
November
December
January
February
March
April
May
June
July
August
September

Fig. 5


Dry Prong Deep Creek at Reservoir near Mercury, Texas
Storm of October 3-4, 1953
Drainage Area 4.32 sq.mi.
Discharge, hundreds of cubic feet per second
Rainfall and runoff in inches
7.74"
Accumulated rainfall, inches
Discharge into reservoir, cfs
3.07"
Accumulated runoff, inches
Discharge from reservoir, cfs
6PM 9 12PM 3 6AM 9 12N 3
Oct. 3, 1953
Oct. 4, 1953

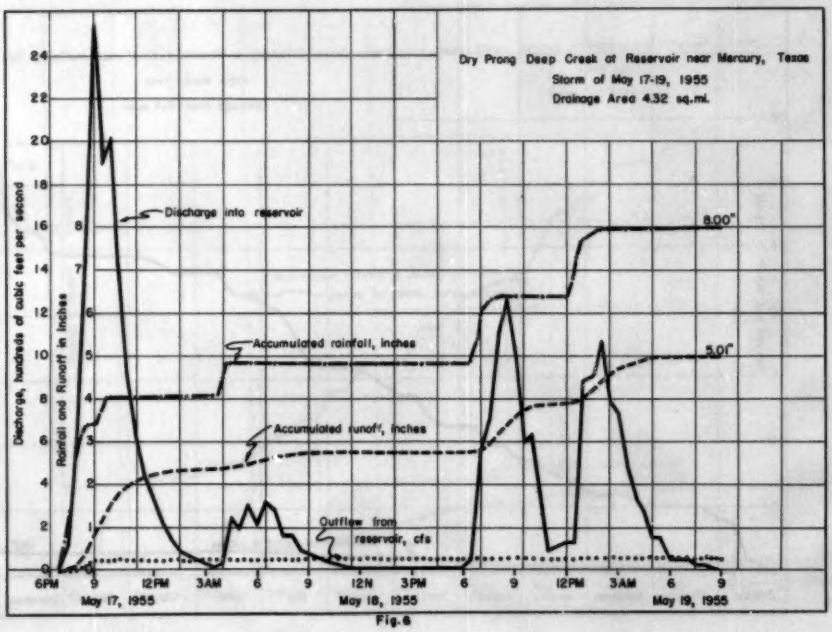

Dry Prong Deep Creek at Reservoir near Mercury, Texas
Storm of May 17-19, 1955
Drainage Area 4.32 sq.mi.
Discharge, hundreds of cubic feet per second
Rainfall and Runoff in Inches
Discharge into reservoir
8.00"
Accumulated rainfall, inches
5.01"
Accumulated runoff, inches
Outflow from reservoir, cfs
6PM 9 12PM 3AM 6 9 12N 3PM 6 9 12PM 3AM 6 9
May 17, 1955
May 18, 1955
May 19, 1955

Fig. 6

Paper 1051 SA 4

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Proceedings of the American Society of Civil Engineers

RADIOACTIVE SEDIMENTS IN THE TENNESSEE RIVER SYSTEM

J. M. Garner, Jr.,[1] and Oscar W. Kochtitzky,[2] A.M. ASCE

SYNOPSIS

This paper discusses the development and use of instruments and techniques to survey gamma radioactivity in the sediments on the bottom of rivers and lakes. The equipment was used and refined during three summer surveys and has subsequently been applied to routine monitoring of bottom sediments in the lakes immediately downstream from Oak Ridge National Laboratory.

The "flounder", having a counting rate of less than 10 counts per second for uncontaminated sediment, readily detected increases in radioactivity in the bottom sediment resulting from liquid wastes released at Oak Ridge National Laboratory. The concentrations of radioactivity in the sediment which were presumed to have originated at Oak Ridge dropped off materially after the first 20 miles of the receiving reservoir and approached background levels at a distance of 100-150 miles. The survey, which extended from above Oak Ridge to the Ohio River almost 600 miles away, revealed several locations where sediment contained radioactivity of natural origin. This had resulted from the erosion of uranium bearing Chattanooga Shale.

No hazardous concentrations of radioactivity of either waste borne or natural origin were found.

INTRODUCTION

The uranium pile and chemical separation plant at the Oak Ridge National Laboratory (originally Clinton Laboratories constructed in 1943 near the Clinch River in East Tennessee) were built to serve as a pilot plant for the Hanford Works in Washington. Since it was expected that these facilities would be operated for only one year, underground tanks were provided for storing the entire anticipated accumulation of radioactive liquid wastes.

The original facilities have continued in use long beyond the pilot plant period and have been increased to the extent that Oak Ridge National

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1. Health Physicist, Oak Ridge National Lab., Oak Ridge, Tenn.
2. Public Health Engr., Tennessee Valley Authority, Chattanooga, Tenn.

Laboratory is not only the principal production center for radioisotopes but is a rapidly expanding reactor research area. With the increase in volume of liquid wastes, there was a modification of the policy from "concentrate and contain" to one providing for the release of some radioactive liquid wastes by "diluting and dispersing" into the natural (and artificially impounded) surface waters of the region.

The maximum permissible concentration (MPC) of radioisotopes of unknown identity in drinking water which could be used throughout one's lifetime without resulting in provable harm[1] is 1×10^{-7} microcuries per cubic centimeter.[3] Releases to the Clinch River are regulated so that the resultant average concentration is kept below this MPC. To facilitate this dispersal there have been constructed a 1,600,000-gallon settling basin for temporary holdup and a 44-acre lake[2] for retention of the wastes before release to the Clinch River. The discharge of radioisotopes into White Oak Creek is limited to 35 curies per week.[3] In terms of the units ordinarily used in water works practice this quantity is extremely small. For example, if radiostrontium, Sr^{89}, were discharged at this rate and uniformly dispersed into a stream flowing 3,500 cubic feet per second, the corresponding concentration would be 21×10^{-12} parts per million and it would take 433 years to dispose of one ounce of strontium, if decay were not taken into account.

Purpose

Radioactive materials are known to be absorbed by biological organisms, adsorbed on clay particles occurring as turbidity in streams, or precipitated by other physical or chemical means thus resulting in the accumulation of radioactive substances in the sediment. The purpose of the surveys was to determine: the extent of dispersion of radioactive materials in river sediment; the level of radiation encountered and consequent external hazard to humans; the amount of present accumulations and predicted future accumulations; the capacity of the TVA system of lakes as a place of permanent disposal; and the validity of the present policy of discharging to nature some of the radioactive wastes.

Procedure

Instrumentation

Due to the short range of alpha and beta particles in water and in mud any potential external hazard that might be encountered in the sediment would most likely be due to gamma emitters. The instrument, called the "flounder", was developed and used in the surveys for measuring gamma radiation in the water and at the surface of the mud.

The flounder is a gamma detector containing twelve thin-wall Geiger-Mueller tubes in a water proof housing 15-1/2" x 15-1/2" x 1-1/2". Figure 1 shows two views of the flounder. The one on the right shows the method of suspension by a yoke clamped to the rubber covered cable. On the left the 1/4" lucite side (treated with aquadag for exclusion of light) is removed to

3. One microcurie is 10^{-6} curies. One curie is the quantity of any radioactive nuclide in which the number of disintegrations per second is 3.7×10^{10}.

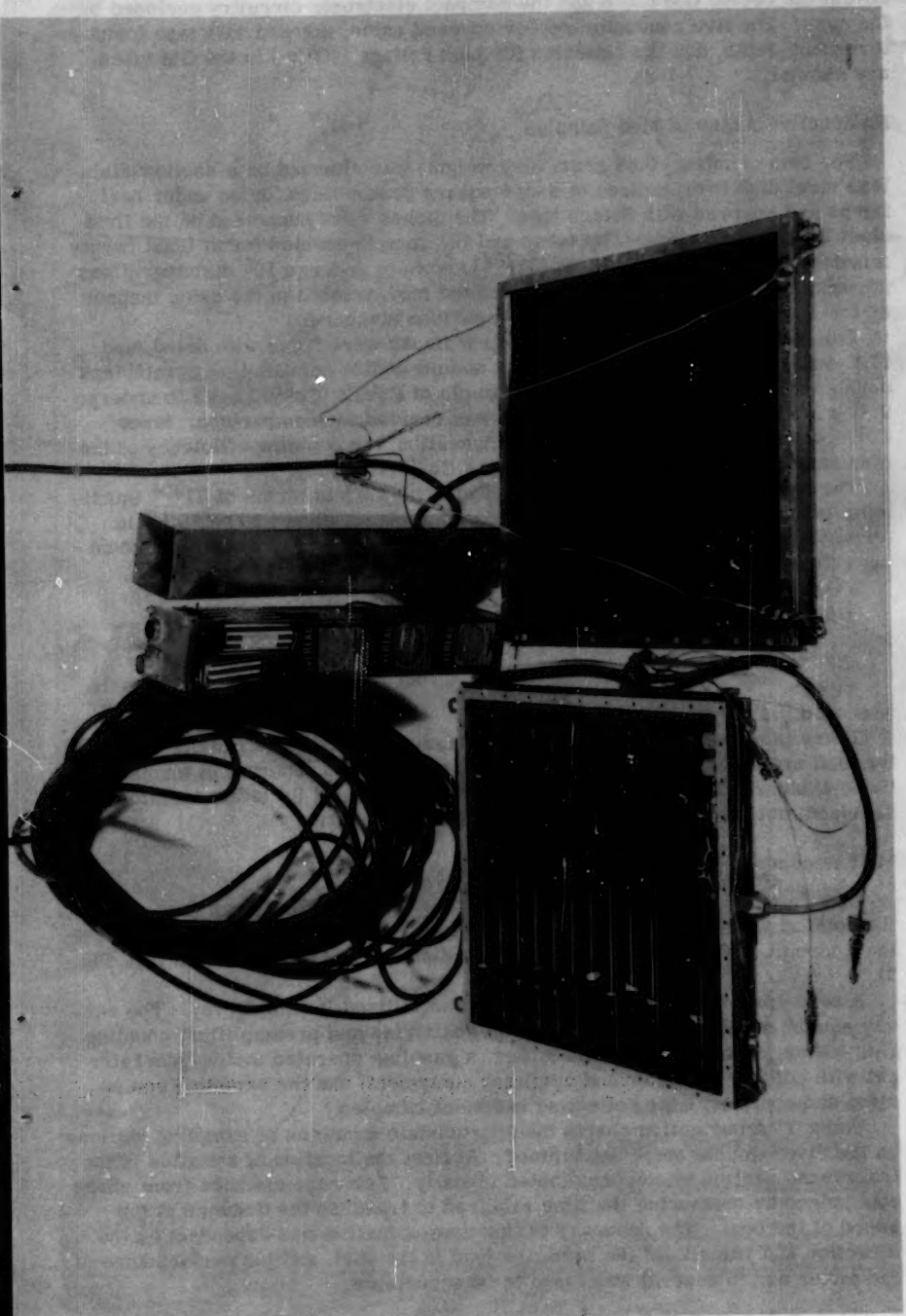

FIG. 1

reveal the twelve GM tubes and the compact electronic circuitry enclosed by the case. The five conductor rubber covered cable, marked with tape for determining depth, and the batteries for high voltage (900 v.) to the GM tubes are shown.

Radioactive Assay of Mud Samples

For beta counting, 0.64 gram (dry weight) was slurried on a shallow stainless steel dish over an area of 6-3/4 square centimeters, dried under heat lamps and covered with Scotch tape. The dishes were supported on the first shelf beneath end window GM tubes and the sample counted for at least twenty minutes. A solution of thallium (Tl^{204}) known to undergo 10^5 disintegrations per minute was mixed with uncontaminated mud, treated in the same manner as the mud sample, and used as a comparison standard.

For gamma counting, 2 ml volumetric flasks were filled with dried mud (2.1 gm), placed within the cavity of a sodium iodide scintillation counter and counted for ten minutes. A two-ml sample of Cobalt (Co^{60}) known to undergo 2.11×10^5 disintegrations per minute was counted for comparison. Since Co^{60} emits two gamma rays per disintegration, the counting efficiency of the mud sample was taken to be one-half the Co^{60} efficiency.

The beta counting efficiency was found to be 5.0% in terms of Tl^{204} (maximum beta energy 0.78 Mev), the gamma counting efficiency to be 16.5% in terms of 1/2 Co^{60} (average gamma energy 1.25 Mev). For converting counting rate into micro-microcuries per gram the formula

$$\mu\mu c/g = \frac{\text{counts per minute}}{\text{counting efficiency}} \times \frac{1}{\text{grams of sample}} \times \frac{1}{2.22 \begin{array}{l}\text{disintegrations} \\ \text{per minute per} \\ \text{micro-microcurie}\end{array}}$$

was used.

There is no reason to believe that the radiations from the radioisotopes in the mud are of these energies. However, activity was reported in terms of these standards (a) to obtain an order of magnitude and (b) to have a simple standard method of reporting activity of unknown energy.

1951 Reconnaissance

Observations were made from July to September in the Clinch River below the point at which White Oak Creek enters and in the TVA reservoirs to about the midpoint of Guntersville Reservoir, a distance of over 200 miles (see Fig. 2).

A seventeen-foot outboard motorboat was utilized in this survey. The survey equipment consisted of the flounder, batteries and preamplifier, counting rate meter, continuous chart recorder, a gasoline operated motor-generator set with voltage regulator and auxiliary equipment, and the sampling equipment and containers for collecting sediment samples.

Using TVA navigation charts the approximate locations of sampling stations on the river or lake were determined. At first the location of a station in the transverse section was approximated visually. Later the distance from shore was judged by measuring the time required to traverse the distance at top speed of the boat. The accuracy of this approximation was dependent on the direction and velocity of the wind, the load in the boat, and the performance of the motor and in general was found to be acceptable.

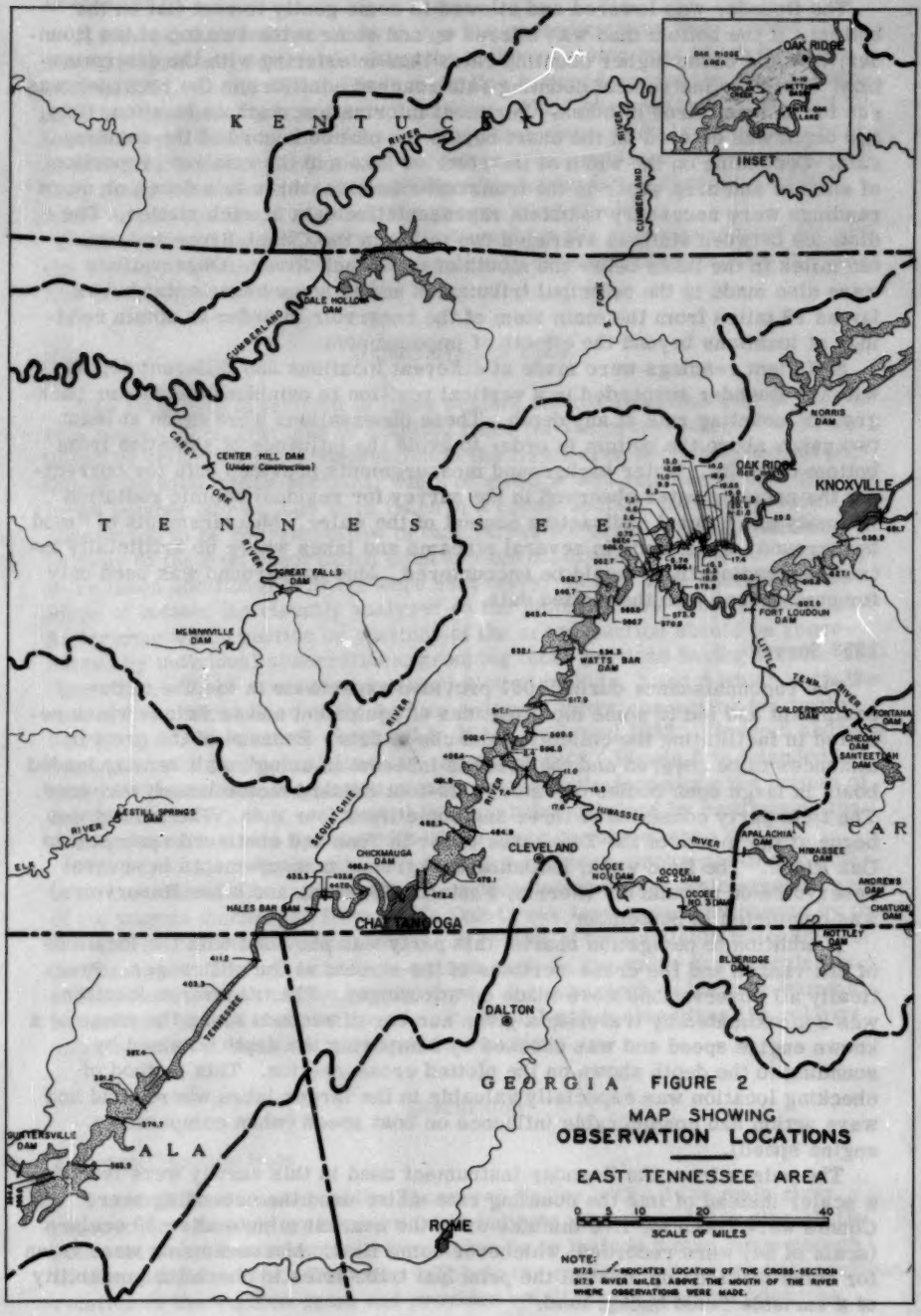

FIGURE 2

MAP SHOWING
OBSERVATION LOCATIONS

EAST TENNESSEE AREA

The flounder was lowered and allowed to come gently to rest flat on the bottom. If the bottom mud was stirred up and some settled on top of the flounder, it would cause higher counting rates thus interfering with the determination. After the instrument counting rate reached equilibrium the recorder was run for at least three minutes. Pertinent information, such as location, time, and depth was entered on the chart beside the plotted record of the counting rate. Depending on the width of the river or lake and the relative proportion of shallow and deep water in the transverse section, three to a dozen or more readings were necessary to obtain representative data at each station. The distance between stations averaged two miles in the Clinch River and nearly ten miles in the lakes below the mouth of the Clinch River. Observations were also made in the principal tributaries and in some cases extended as far as 19 miles from the main stem of the reservoir in order to obtain readings at locations beyond the effects of impoundment.

Sufficient readings were made at different locations and different depths with the flounder suspended in a vertical position to establish the "water background" counting rate at any depth. These observations were made at least two yards above the bottom in order to avoid the influence of radiation from bottom deposits. Water background measurements provided data for correcting the counting rates observed in the survey for residual cosmic radiation intensity and natural radioactive content of the water. Measurements of "mud background" were made in several streams and lakes where no artificially introduced radioactivity would be encountered. Mud background was used only for comparison with the plotted data.

1952 Survey

The reconnaissance during 1951 provided experience in the use of the equipment and led to some modifications of equipment and technique which resulted in facilitating the collection and use of data. Because of the greater distances to be covered and the hazards inherent in using small heavily loaded boats in large open bodies of water, a 22-foot inboard motor launch was used. The field party consisted of three and sometimes four men. The survey was begun at the mouth of the Tennessee River in June and continued upstream to Oak Ridge. The field work, including background measurements in several reservoirs on tributaries (Norris, Fontana, Hiwassee and other Reservoirs) was completed in September.

In addition to navigation charts, this party was provided with the locations of silt ranges and the cross-sections of the stream at the silt ranges. Practically all observations were made on silt ranges. The transverse location was approximated by traveling a given number of seconds along the range at a known engine speed and was checked by comparing the depth obtained by sounding to the depth shown on the plotted cross-section. This method of checking location was especially valuable in the larger lakes where wind and wave action had considerable influence on boat speed (when compared to engine speed).

The pulses from the flounder instrument used in this survey were fed into a scaler instead of into the counting rate meter used the preceding year. Counts were taken for five minutes or to the nearest minute after 30 scalers (scale of 64) were recorded, whichever came first. Measurements were taken for some distance up most of the principal tributaries to check the possibility of a variable "mud background."

1953 Survey

A less extensive survey than that in the preceding year was carried out during August and September using the outboard motor boat which had been used in 1951. In the Clinch River a tag line fastened across the stream was used for determining transverse locations. More observations (at 50-foot intervals) were made in the cross-sections than were made in the same cross-sections during preceding years, but fewer cross-sections were investigated. Only three cross-sections in Watts Bar Reservoir (other than the Clinch River embayment) and two in Chickamauga were studied.

A battery powered scaler was substituted for the motor-generator set and the scaler previously used. This resulted in considerable reduction in weight and crowding in the boat.

Presentation of Data

The total data assembled in these studies are too voluminous to include in this report. The averages of counting rates for the cross-sections were obtained by weighting each observation in the transverse section by the proportion of the total cross-sectional width of which it was representative. When a large number of readings were taken at a cross-section, each observation was considered to be representative of that part of the cross-section extending halfway to the next observation point in each direction. When fewer readings were taken additional criteria were necessary. In this case both depth and slope of bottom (as visually analyzed on the plotted cross-section) were used in deciding which portion or portions of the cross-section should be represented by individual observations, grouping those portions having similar configuration. These averages have been plotted in Figs. 3 and 4 which show the averages at the surface of the bottom sediment (after correcting for "water background") which were observed in 1951, 1952, and 1953. Figure 5 shows a comparison of the highest counting rates observed. Figure 6 is intended to compare the total accumulation of radioactive sediments along the bottom of the Clinch River; on it are plotted the quantities obtained by multiplying the average counting rate by the width of the river or lake at each station.

Efforts to identify the particular gamma emitters discharged and to determine the ratio of the rate of gamma-ray emission to total disintegration rate of the wastes discharged from White Oak Creek have been handicapped by the low concentrations of radioisotopes. A review of the available records[5] in an attempt to determine the beta-gamma ratio in the water has revealed inconsistencies which cannot be explained. A comparison of the beta-gamma ratio found in mud samples is given in Table I which also gives the relative counting rate of samples from various locations.

Discussion

Average Counting Rate

Figure 7 shows a map of the Tennessee Valley with all the main river reservoirs. It would be impractical to indicate on a map of this size all the points at which measurements were taken or to include a map large enough to show all the points. This map will help the reader to visualize the locations relative to the various dams and provides relative geographical positions.

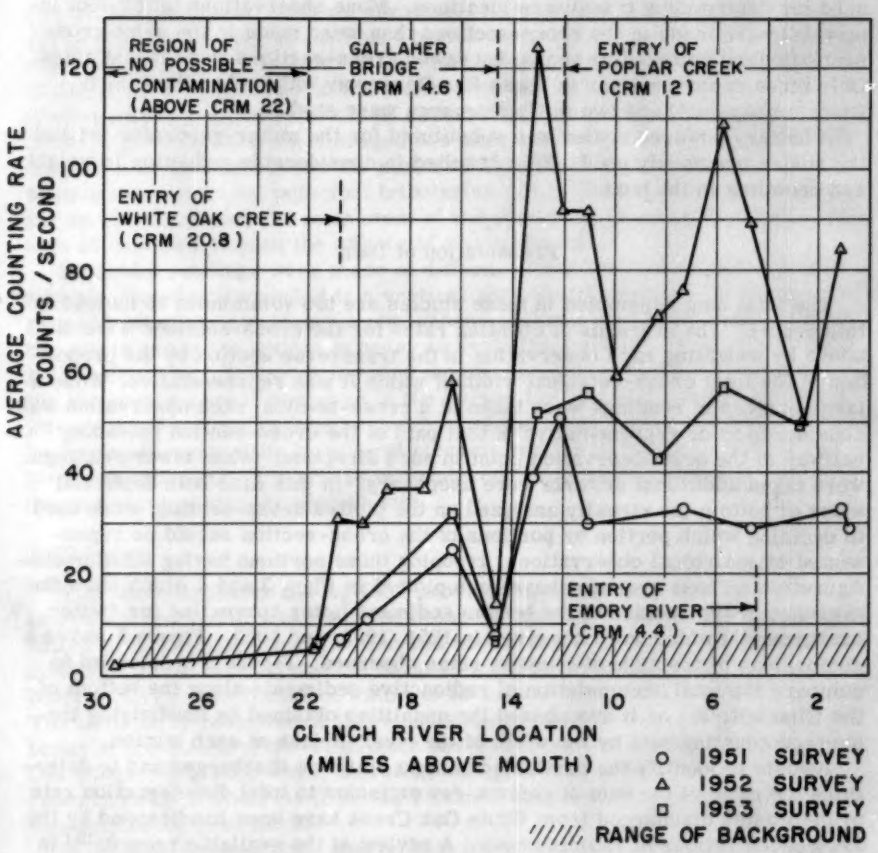

FIGURE 3
CLINCH RIVER SEDIMENTS
AVERAGE RADIATION COUNTING RATE AT MUD SURFACE

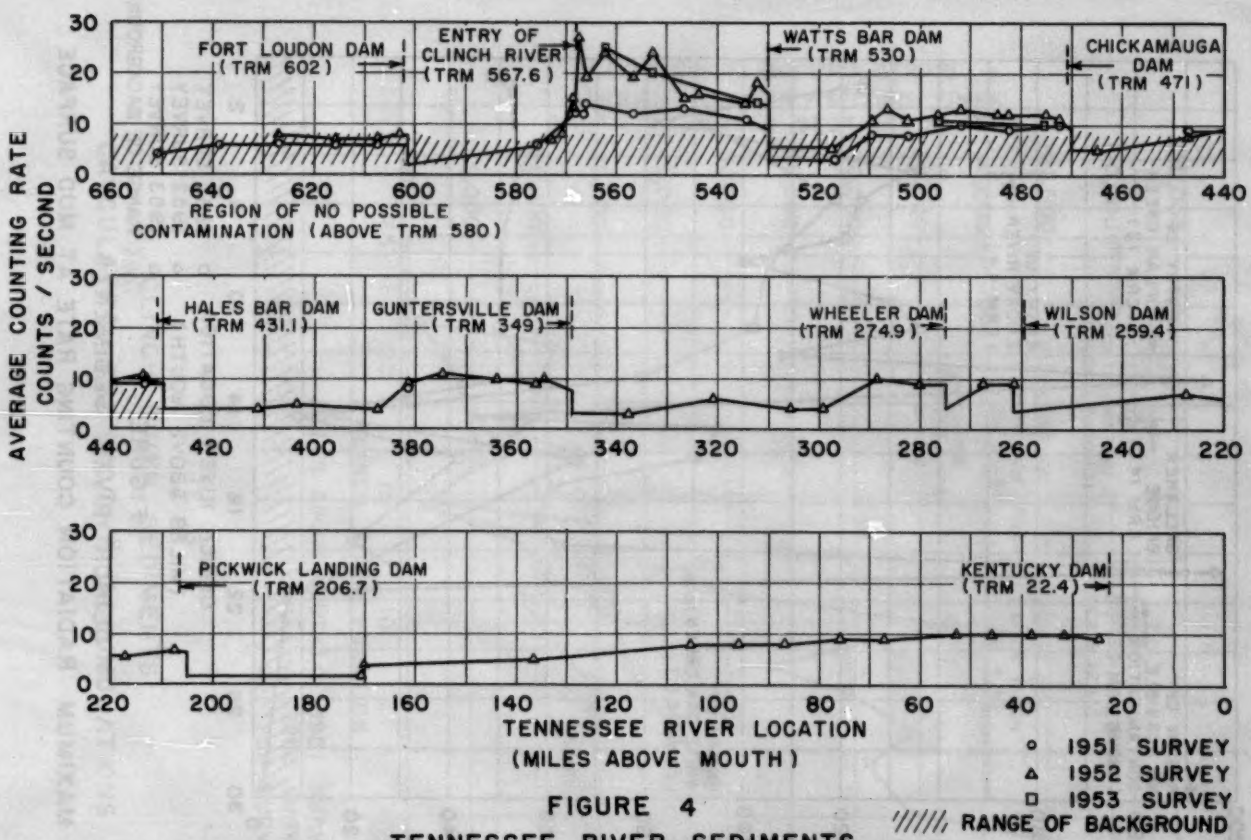

FIGURE 4

TENNESSEE RIVER SEDIMENTS

AVERAGE RADIATION COUNTING RATE AT MUD SURFACE

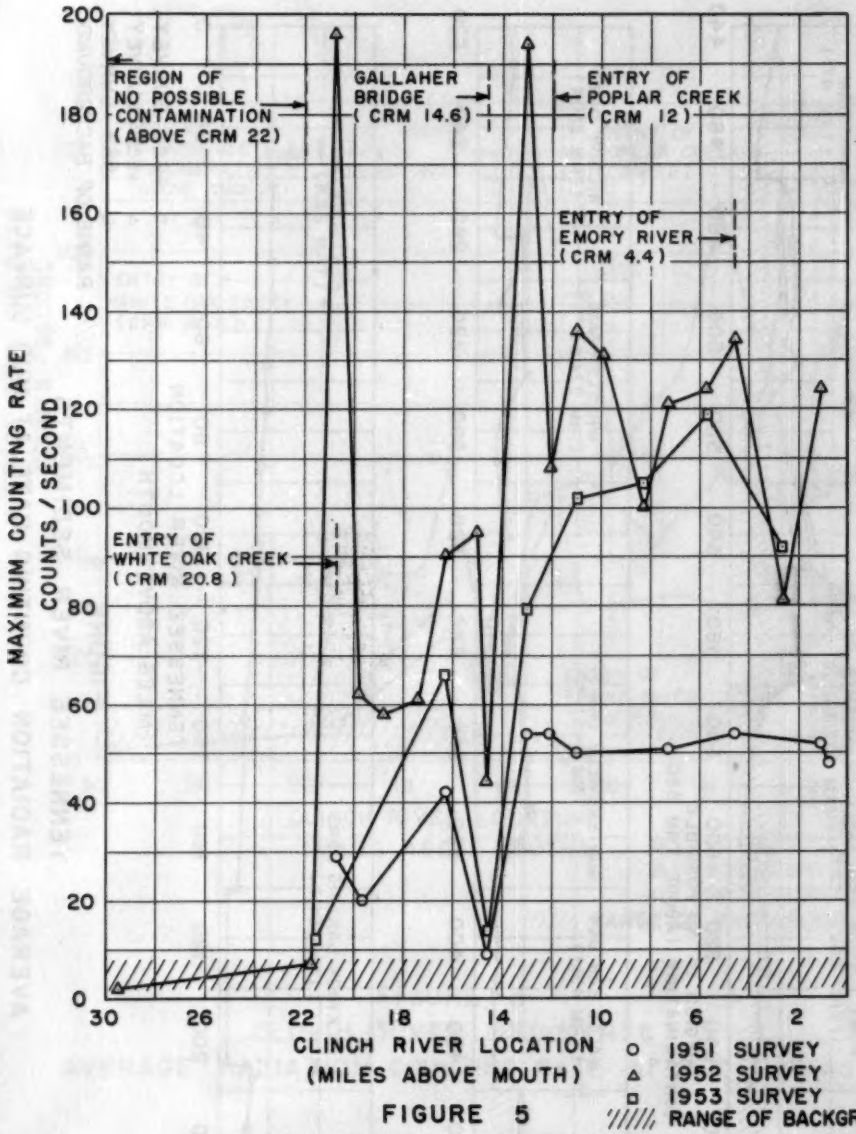

FIGURE 5

CLINCH RIVER SEDIMENTS

MAXIMUM RADIATION COUNTING RATE AT MUD SURFACE

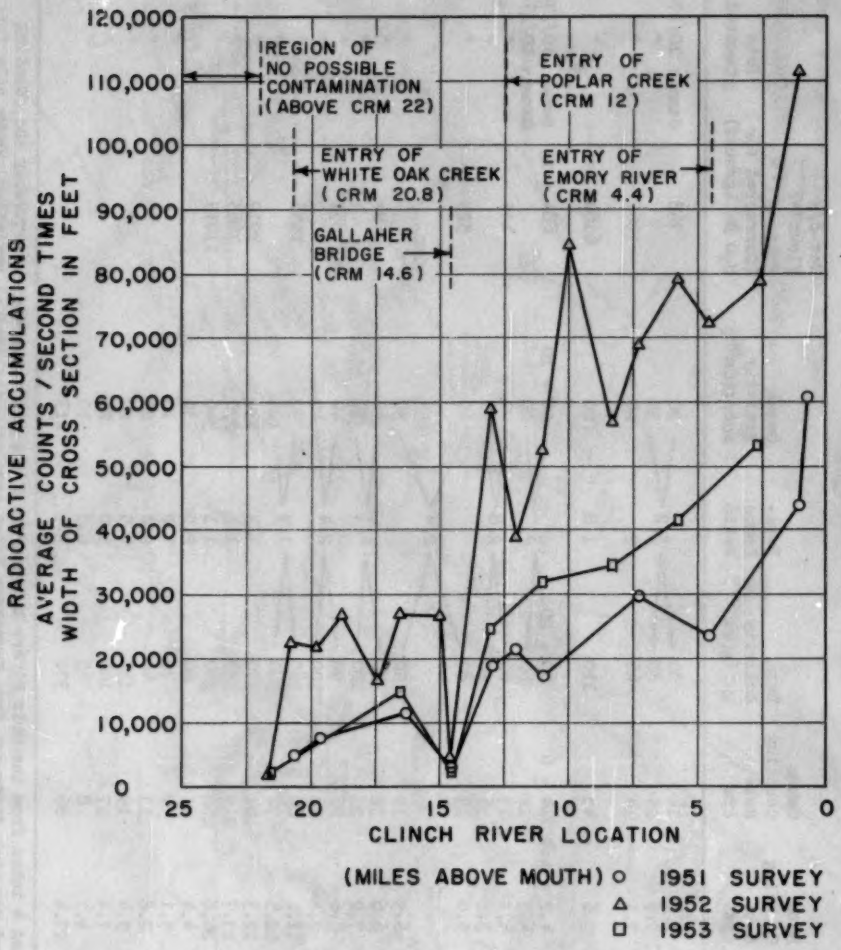

FIGURE 6

CLINCH RIVER SEDIMENTS

CALCULATED RADIOACTIVE ACCUMULATIONS

Table I

Radioassay of Mud Samples

Sample No.*	Location	Beta Counting Rate cpm	Gamma Counting Rate cpm	Beta Activity $\mu\mu c/g(Tl^{204})$	Beta-Gamma Ratio	Gamma Activity $\mu\mu c/g(Co^{60})$	Net c/m Flounder Reading (Corrected for H_2O Background)	Date Counted
2A	ORM 937.3	5.4	14	80	3.2	20	748	Gamma: 10/17-23/52
2B	(below Tenn.R)	5.5	25	80		30		"
3	ORM 930.2 (above Tenn.R)	3.1	14	40	2	20	1866	"
4	ORM 928.5 (below Cumb.)	7.6	48	110	1.8	60	6184	"
1A	TRM 45.8	4.2 ± 1.4	14 ± 6	60 ± 20	3	20 ± 10	684 ± 23	Beta: 10/17-18/52
1B		4.3	12	60		20		Gamma: 10/17-23/52
5A	TRM 207.3	4.2	17	60	2.8	20	455	"
5B		3.8	16	50		20		"
7A	Wilson 4 (mi. 267.4 4000 ft.)	5.6	27	80	2.4	40	525	"
7B		6.3	19	90		30		"
8A	Wheeler 8	6.6	32	90	2.7	40	594	"
8B	(TRM 280.0)	7.2	21	100		30		"
9A	Guntersville	6.6	28	90	2.4	40	851	"
9B	(TRM 374.7)	7.1	30	100		40		"
13A	TRM 552.7	11.1	69	160	1.9	90	1956	"
13B		12.5	76	180		100		"
14	TRM 567.4	12.3	101	170	1.3	130	3219	"
15	CRM 9.9	23.7	219	330	1.1	290	7865	"
16	CRM 13.0	26.8	260	380	1.1	340	11615	"
18	Big Creek	4.4	21	60	2.0	30		11/7/52
21	Powell R.	5.1	9	70	7.0	10		"
23	Clinch R.	2.8	13	40	2.0	20		"
24	Hiwassee R.	9.4	19	130	6.5	20		"
25	Ocoee R.	7.4	17	100	5.0	20		"
26	Santeetlah	4.1	8	60	6.0	10		"
Chatt.	Shale/Jackson Co.	27.4	86	370	3.3	113		

Note: For Samples 3 and 4 taken from the Ohio River, the energy of the radiation and consequently the counting efficiency in mud was much different from Tennessee River samples. Statistical variation, shown only for Sample 1A, indicates magnitude of probable errors.

* A and B denote duplicate assays for the same sample.

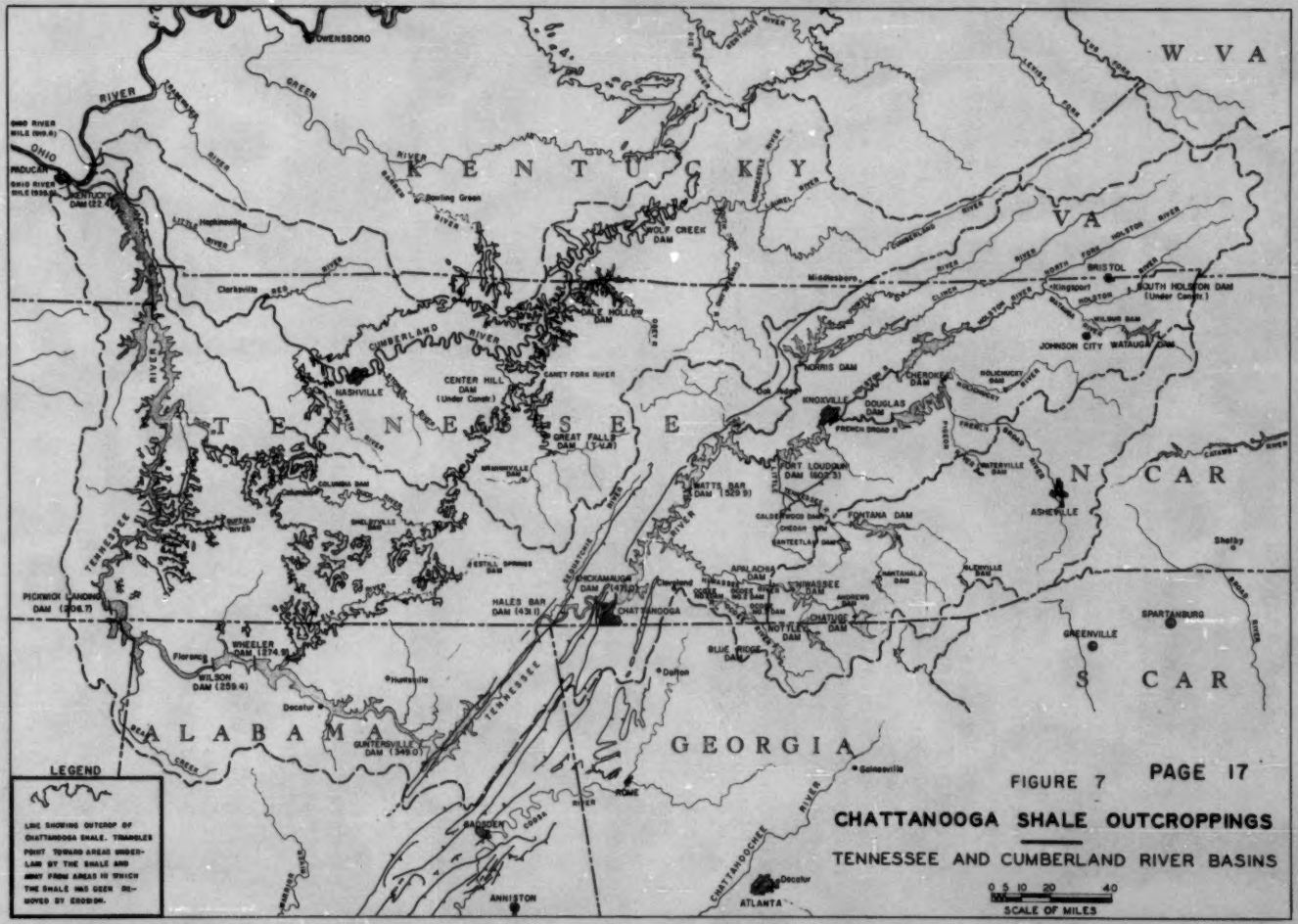

FIGURE 7

CHATTANOOGA SHALE OUTCROPPINGS

TENNESSEE AND CUMBERLAND RIVER BASINS

The Cumberland River Valley, just north of the Tennessee Valley, is also included. The general location of outcroppings of Chattanooga Shale has been indicated.[6,7]

Wastes which have been stored in White Oak Lake until discharge enter the Clinch River arm of Watts Bar Reservoir at Clinch River Mile 20.8. While backwater from Watts Bar Dam extends 28 miles up the Clinch River at times of full pool, it barely extends to the mouth of White Oak Creek at minimum pool level (October-April). Thus, for much of the year considerable velocity occurs in the Clinch River at the mouth of White Oak Creek. This velocity tends to scour the sediment which might otherwise be deposited at this point. Referring to Fig. 3 a gradual buildup below Mile 20.8 is seen which is interrupted by scour at Mile 14.6. At this point the channel is restricted by the Gallaher Ferry bridge and the water velocity prevents the deposition of bottom sediment. The increase in counting rate continues downstream to Mile 12 in 1951, Mile 13 in 1952, and Mile 11 in 1953. From there to the mouth of the Clinch River the counting rate was relatively constant in 1951 and 1953 and of the same order of magnitude, though not quite so constant, in 1952. A phenomenon of deep reservoirs known as thermal stratification[8,9] may be responsible for this deposition pattern. During the summer the water released at Norris Dam is considerably colder than the water on the surface of Watts Bar Reservoir. Due to density differences this cold water "ducks under" the warm water on the surface and flows as a bottom stratum. It can readily be seen that the cold water will comprise practically all the water moving downstream in the Clinch River embayment and the velocity and consequent carrying capacity will vary accordingly.

The counting rates observed in 1952 were approximately twice those measured in 1951 and 1953. Referring to monitoring data[5] there seems to be some indication of a shift in the relative predominance of various radioisotopes in the wastes before the 1952 survey. This apparent trend cannot tell the whole story since the elapsed time between sample collection and radiochemical analysis (in the routine monitoring program) would permit the reduction by decay of such short lived isotopes as barium. While the barium in the various component parts of the composite sample might have passed through one to three half-lives, barium still accounted for as much as 20 percent of the identified beta radioactivity during one month. Freshly discharged wastes of short half-life, such as barium, might have been detected in the stream bed by the flounder and still might decay to the vanishing point during the six weeks required for collection and analysis of the monthly composite sample. It would have decayed below detectable concentration by the time of the 1953 survey.

During the summer (June-September) of 1952, stream flow in the Clinch River[10] was less than 60 percent as great as the flow during the corresponding period in 1951 or 1953. With low flow the stream's capacity to transport suspended matter is reduced. Consequently, a larger portion of the transported silt was deposited in the Clinch River embayment during 1952 than during 1953.

On Fig. 3 it is seen that the curve for radiation counting rate in the Clinch River sediment during 1952 is approximately twice as high as that for 1953. On the other hand in Watts Bar Reservoir proper (Fig. 4) there is little difference observed between the two years.

By analysis it was determined that the naturally occurring radioactive isotopes of uranium, thorium, and potassium accounted for only 25 percent of

the radioisotopes in the Clinch River mud (Mile 13) and 43 percent in the Tennessee River sample (Mile 567.4). The remainder of the radioactivity was from fission products of which the most prominent were ruthenium, cesium, and cerium.

An interesting situation was encountered during 1952 in a stretch of the Clinch River between Mile 4.4 where the Emory River enters and Mile 0.0 where the Clinch flows into the Tennessee River. The radioactivity just below the Emory River was observed to be lower than that either above or below this stretch. At this location the TVA initiated construction activities at the Kingston Steam Plant and it is concluded that mud washed in from the construction area covered and shielded from the flounder much of the radioactive sediment that had previously been deposited.

Immediately upon passing out of the Clinch River into the main stem of Watts Bar Reservoir the radiation levels measured were much lower than in the Clinch River (see Fig. 4). This was to be expected because of the greater expanse over which the sediments were deposited. Counting rates dropped off progressively downstream, but here again as in the Clinch River the 1952 and 1953 levels were higher than the 1951 values. For several miles below each dam only background radiation was detected. This resulted from the absence of sediment in the turbulent reaches below the dams.

The highest intensities detected at various stations in the Clinch River are plotted on Fig. 5.

Total Radioactive Sediment

Referring to Table II[11] it is seen that of the sediment in Chickamauga Reservoir only 23 percent is expected to have come from above Watts Bar Dam. Of that in Hales Bar only 29 percent originated above Chickamauga and by extending the calculation, only 6.7 percent from above Watts Bar Dam. Passing successive downstream dams the proportion of sediment in the various reservoirs originating above Watts Bar Dam would be Guntersville 2.0 percent, Wheeler 0.28 percent, Wilson 0.15 percent, Pickwick 0.06 percent, and Kentucky 0.01 percent. Since on a drainage area basis only 34 percent of the sediment entering Watts Bar Reservoir would be considered to have originated in the Clinch River basin, it may be seen that very small proportions of the sediment in the downstream reservoirs could be expected to have originated in the Clinch basin.

An effort has been made to approximate the amount of radioactive materials in the sediment. Since it was not possible to determine the depth of radioactive sediment or to obtain an undisturbed bottom sample for such a determination, it was necessary to make some assumptions. The first assumption was that the radiation detected by the flounder represented all the radioactivity in the sediment. The second was that such a measurement was characteristic of the sediment in certain portions of the cross-section as previously determined (see Presentation of Data). Both of these assumptions may be subject to great error.

Quantities proportional to the total radioactive sediment were calculated for all cross-sections measured in the river system by multiplying the average counting rate in each cross-section by the width of the stream at that cross-section. The values determined in this manner for the Clinch River embayment are plotted on Fig. 6. Whereas counting rates tend to level off as the mouth of the Clinch River is approached and to drop considerably after

Table II

Estimated Origin of Sediment

Reservoir	Source of Sediment in Percent of Total: From Principal Tributary Area	From Other Local Area	From Above Next Upstream Dam
Kentucky	53	28	19
Pickwick	41	19	40
Wilson	32	16	52
Wheeler	73	13	14
Guntersville	36	34	30
Hales Bar	33	38	29
Chickamauga	49	28	23
Watts Bar, less Clinch	31	26	43
Clinch Embayment	60	40	0

The Principal Tributary Area in each case includes all tributaries of 200 square miles or larger.

reaching the main stem of Watts Bar Reservoir, the total accumulation continues to build up and the point of maximum accumulation of radioactive material (not shown on Fig. 6) seems to be about Mile 552.7 on the Tennessee River. Surprisingly enough, total accumulations in various cross-sections of Guntersville and Kentucky Reservoirs are almost as great as in Watts Bar. When considered with the length of river over which such accumulations were observed, it appeared that some other source of radioactive material must be responsible for this occurrence. Furthermore, during the 1952 survey counting rates in the Ohio River near the mouth of the Tennessee and in the mouth of the Cumberland River were found to be higher than those observed in the Tennessee River a short distance above its mouth.

Radioactive Sediment of Geological Origin

There were no atomic energy installations on the Cumberland River to account for this radioactivity and no large research centers which might have accidentally lost such large quantities of radioactive materials. Chattanooga

Shale[7], a uranium bearing rock of the Devonian formation, outcrops extensively on the Cumberland Plateau (see Fig. 7) where it is for the most part flatbedded. Most of the outcroppings in the Tennessee Valley are tilted steeply and consequently result in less dissolution of radioactive constituents. While a narrow band of Chattanooga Shale parallels the Tennessee River above Chattanooga and narrow bands cross the Tennessee River at Chattanooga, there are few deposits of this shale outcropping in east Tennessee. In traveling downstream the first major outcrop is found between the Sequatchie River and the ridge which parallels it to the east. The Sequatchie River flows into the Tennessee River a short distance below Hales Bar Dam where for several miles turbulence would prevent extensive deposition. The counting rates in Guntersville Reservoir begin to increase downstream from Mile 390 and reach a level which cannot reasonably be expected to have been caused by materials originating at Oak Ridge since less than 1 percent of these sediments originated in the Clinch River basin. The only reasonable conclusion is that this results from erosion or leaching of radioactive substances from the Chattanooga Shale in the Sequatchie Valley and other local deposits.

In Wheeler Reservoir the rise in counting rate detected below the Elk River is undoubtedly the result of erosion of the outcropping of Chattanooga Shale in that section of north Alabama and south central Tennessee.

In Kentucky Lake the reach of the river in which the counting rate increases was not identified as well as might be desired, but there should be no doubt of its correlation with the Chattanooga Shale deposits in the Duck and Buffalo River areas.

Referring to Table I it is seen that the beta-gamma ratio ($\mu\mu$c/gm beta divided by $\mu\mu$c/g gamma) in the Clinch River mud was 1.1. In Watts Bar Reservoir at the mouth of the Clinch River it was 1.3 and a short distance farther downstream 1.9. In Guntersville Reservoir it was 2.4, Wheeler, 2.7, Pickwick 2.8, and Kentucky 3.0. By comparison the beta-gamma ratio from Chattanooga Shale was 3.3. Thus, the beta-gamma ratio from bottom sediments is seen to increase (approaching that of Chattanooga Shale) as the prevalence of shale in the watershed increases.

The counting rates observed in the Ohio River and in the mouth of the Cumberland River are believed to be attributable to the extensive outcroppings of Chattanooga Shale in that watershed.

Possible Exposure to Radiation

The natural background exposure rate for man averages from 0.01 - 0.1 mr/hr[4] from cosmic and other natural radiation.[12] The Recommendations of the International Commission on Radiological Protection[13] are "in circumstances under which the whole body may be exposed over an indefinite period to X or gamma radiation of quantum energy less than 3 Mev the maximum permissible dose received by the surface of the body shall be 0.5r in any one week. This dose corresponds to 0.3 r/week measured in free air." Since the highest counting rates corresponded to 0.14 mr/hr the dose which might result from reclining on the mud continuously would amount to only 0.024 r/week or 8 percent of the maximum permissible dose.

While no one could be expected to lie on the bottom sediments continually, it is conceivable that the sediments might be dredged from the bottom and used in such a manner as to expose humans to the radiation. In this regard

4. Milliroentgens per hour.

an employee in a sand handling plant might be expected to be closer to the radioactive material for a longer period than any other person. If he were employed eight hours a day five days a week in such a place that he would receive 50 percent as much radiation as if he were lying directly on the lake bottom, his exposure for the week would amount to only 12 percent of that possible from lying on the bottom. Radiation would have to be increased to 104 times its 1952 level before such an individual would receive a maximum permissible dose. Assuming that the rate of annual accumulation as indicated by the increase observed from 1951 to 1952, is equal to half of the 1952 total, and might continue at a uniform rate in the future it would be 208 years (disregarding decay) before radiation would approach a level which might possibly result in significant exposures. When radioactive decay is taken into consideration it can be seen that such an exposure would never be encountered since the average half-life of the mixture of radioisotopes in the wastes would be much less than 33 years, which is the half-life of Cs^{137}, the longest lived isotope of those in the laboratory wastes for which analysis is routinely made. Thus, at this assumed rate of accumulation of radioactive sediments containing isotopes of half-lives comparable to the present known constituents the concentration would never reach a level which would create a human hazard from external exposure.

CONCLUSIONS

It is believed that enough experience has been gained with various types of equipment to indicate that the procedures outlined herein are universally applicable to stream surveys.

It is believed that dispersion of radioactive sediment resulting from waste discharged from Oak Ridge does not extend downstream farther than Chickamauga Dam, or possibly Hales Bar Dam, to any measurable extent. The principal part of radioactive sediments found in Guntersville Reservoir and at points farther downstream are believed to have been derived from the erosion or leaching of Chattanooga Shale deposits.

No external exposure hazard has developed beyond the area of control as a result of the ORNL waste discharge program and the level of radiation would have to be over twelve times as great as the 1952 survey revealed before any hazard, however improbable, would even be possible. Since turbidity would normally be removed by conventional water treatment processes and the concentration of radioisotopes in solution is below the maximum permissible level for long time use, there is no ingestion hazard to humans resulting from waste disposal practices at the laboratory and the present water uses of the streams.

If the quantities and types of wastes discharged from ORNL were to continue at the same rate as during the year 1951-1952 and there were no other large sources of radioactive wastes developed in the same vicinity, the TVA lake system could store the resultant radioactive sediments indefinitely without presenting any external exposure hazard to humans.

On the other hand, from 1951 to 1952 to 1953 the calculated average concentration of radioactive substances in the Clinch River below the mouth of White Oak Creek rose from about 27 percent to 68 percent to 90 percent of the concentration which is acceptable in drinking water for lifetime use.(5) Should this trend be followed by the discharge of larger quantities of wastes

of greater concentration of radioactivity, the potential internal hazard resulting from the use of the Clinch River as a source of drinking water for the gaseous diffusion plant at K-25 and the Kingston Steam Plant would have to be assessed.

ACKNOWLEDGMENTS

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The "flounder" instrument was developed by J. M. Garner and produced in the research shop of the ORNL Health Physics Division. Related instruments were adapted from commercially available equipment. R. L. Nichols assisted in the operation and maintenance of electronic equipment.

Silt range sections were provided by the Hydraulic Data Branch, TVA, and TVA Navigation Charts were used.

Methods for radioassay or radiochemical analysis were developed or adapted to this study by Bernd Kahn.

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Paper 1053 SA 4

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THE ROLE OF CHEMICALLY COMBINED OXYGEN IN BIOLOGICAL SYSTEMS

Ross E. McKinney,[a] M. ASCE
(Proc. Paper 1053)

Oxygen is the most important element in biological systems, for without oxygen there could be no growth of microorganisms or utilization of organic matter. Sanitary Engineers are familiar with the role of dissolved oxygen in the stabilization of organic wastes by aerobic systems but little is known concerning the role played by chemically combined oxygen in either aerobic or anaerobic systems. Actually, chemically combined oxygen is more important to microorganisms than dissolved oxygen. The purpose of this paper is to show the general relationships of the various forms of combined oxygen to the microorganisms in biological systems.

Water

Water is the most abundant oxygen containing molecule in biological systems. For the most part, water is merely the suspending fluid for the microorganisms and the chemical reactions they produce. But water also is the major source of oxygen in biological oxidation and synthesis reactions.

Oxidation:

Microorganisms must oxidize organic matter either aerobically or anaerobically in order to obtain sufficient energy to synthesize new cells or to remain alive.(1) The primary mechanism of aerobic and anaerobic oxidation is by the coenzyme, diphosphopyridine nucleotide (DPN). The difference between aerobic and anaerobic oxidation is in the final hydrogen acceptor. DPN oxidizes organic matter by the removal of two hydrogen atoms, Fig. 1a, rather than by the direct addition of oxygen. In aerobic systems the reduced DPN, $DPNH_2$, passes the hydrogen to flavinadenine nucleotide, FAD, which passes them to a series of Cytochrome reactions where the hydrogen is combined with dissolved oxygen to form water. In anaerobic systems the $DPNH_2$ gives up the two hydrogens to some oxygen containing compound. In both systems it is essential that the $DPNH_2$ give up its hydrogen and thus be

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a. Asst. Prof. of San. Eng., Massachusetts Inst. of Technology, Cambridge, Mass.

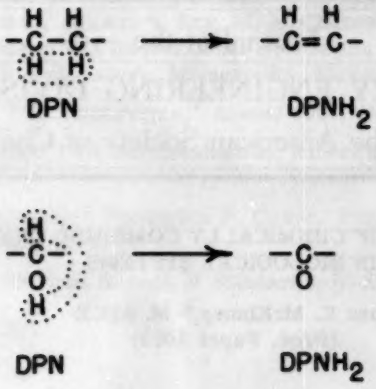

Figure 1a - Hydrogen Removal Mechanisms

Figure 1b - Addition of Water(Oxygen) to Organic Matter

oxidized back to DPN. The amount of DPN within a microorganism is limited and must be continuously regenerated. (Fig. 2)

The removal of hydrogen from the organic compound creates an unsaturated double bond between adjacent carbon atoms. The double bond is highly reactive and water is added to the molecule at this point. It is by this indirect mechanism that oxygen is added to the organic molecules being oxidized. (Fig. 1b)

Synthesis:

Examination of chemical compositions of pure cultures of bacteria shows that the molecular formula for bacterial protoplasm is $C_5H_9O_3N$.(2) Protoplasm is not a single compound but a tremendous number of individual molecules combined to make the over-all protoplasmic structure. The bacteria in waste systems must have all of the basic mechanism to take a simple organic compound and synthesize all of the molecules making up protoplasm. If the compound being metabolised contains more oxygen than required for protoplasm, the excess oxygen is used to oxidize a portion of the compound to secure the energy necessary to produce the protoplasm. If the compound being metabolised contains less than sufficient oxygen for synthesis, the

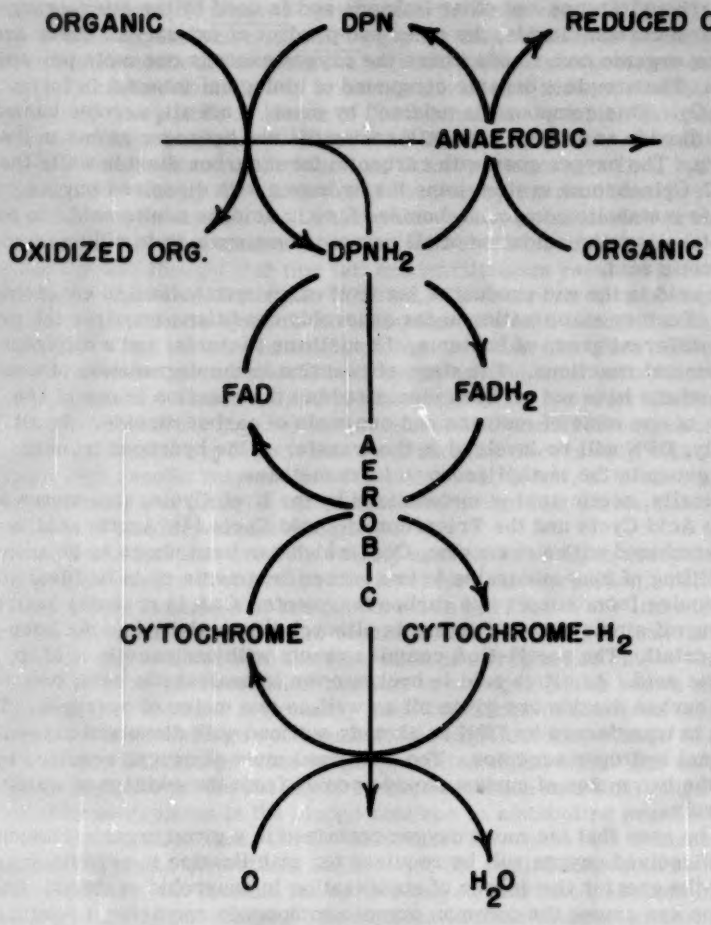

Figure 2 - Hydrogen Transfer Mechanisms

additional oxygen is added from water in the same manner as shown for oxidation. The synthetic and the oxidative metabolic pathways are inter-related with some common portions and some separate portions. It is even possible to reverse the synthesis pathway and use it as an oxidative pathway during endogenous metabolism. The synthesis of some of the components of protoplasm requires different hydrogen arrangements than found on the original organic matter being metabolised. The hydrogen transfer is brought about by DPN with the new structure becoming the hydrogen acceptor, even in aerobic systems.

Organic Oxygen

Oxygen is contained in organic compounds as hydroxyl groups, carbonyl

groups, carboxyl groups and ether linkages and is used by the microorganisms to form carbon dioxide, the other end product of oxidation. There are no common organic compounds where the oxygen exceeds one mole per atom of carbon. The simplest organic compound of biological interest is formic acid, CH_2O_2. This compound is oxidized by most, if not all, aerobic bacteria to carbon dioxide and water with DPN taking the two hydrogen atoms in the initial step. The oxygen goes with carbon to form carbon dioxide while the DPN-FAD-Cytochrome system joins the hydrogen with dissolved oxygen. The other basic metabolic compound, besides formic acid, is acetic acid. In bacterial metabolism the oxidation of all organic compounds is to either formic acid or acetic acid.

Acetic acid is the end product of the first stage metabolism in anaerobic systems. Further stabilization under anaerobic conditions requires the presence of a different group of bacteria, the methane bacteria, and a different set of chemical reactions. The steps of reaction in the degradation of acetic acid to methane have not all been elucidated but the reaction is one of the formation of one mole of methane and one mole of carbon dioxide. In all probability, DPN will be involved in the transfer of the hydrogen from the carboxyl group to the methyl group to form methane.

Aerobically, acetic acid is metabolised by the Kreb Cycle, also known as the Citric Acid Cycle and the Tricarboxylic Acid Cycle.(3) Acetic acid is actually combined with a coenzyme, CoA, which has been shown to be active in the splitting of long molecules to two carbon fragments or in building complex molecules from simple two carbon fragments. CoA is probably active in splitting off single carbon fragments although this activity has not been shown in detail. The acetyl-CoA complex reacts with oxaloacetic acid to form citric acid. As citric acid is broken down to oxaloacetic acid, two moles of carbon dioxide are given off as well as two moles of hydrogen. The hydrogen is transferred by DPN as already outlined with dissolved oxygen being the final hydrogen acceptor. The additional mole of oxygen required to produce the two moles of carbon dioxide comes from the addition of water to the molecule.

It can be seen that the more oxygen contained in a given organic molecule, the less dissolved oxygen will be required for stabilization in aerobic systems and the greater the degree of stabilization in anaerobic systems. Carbohydrates are among the common organic compounds requiring a minimum of extra oxygen. Aerobically, carbohydrates use 1.07 grams of oxygen per gram carbohydrate oxidized. Anaerobically, carbohydrates can be completely oxidized to acetic acid without additional oxygen and then methane bacteria break the acetic acid down to methane and carbon dioxide. Fatty acids are among the common organic compounds requiring the maximum additional oxygen, approximately 3.4 grams oxygen per gram fatty acid in aerobic systems. The lack of much oxygen in the molecule limits anaerobic metabolism and prevents complete metabolism of fatty acids.

Nitrites and Nitrates

Oxygen combined with nitrogen to form nitrites and nitrates is common in aerobic biological waste treatment systems. The stabilization of domestic sewage, protein wastes and high ammonia wastes results in an excess of ammonia nitrogen. In the presence of dissolved oxygen the autotrophic

nitrifying bacteria,(4) Nitroso- group, oxidize ammonia to nitrites. Another group of autotrophic bacteria, Nitro- group, oxidize nitrites to nitrates. The autotrophic bacteria differ from the heterotrophic bacteria which stabilize the organic matter in their use of carbon dioxide as their carbon source for protoplasm. Very little is known of the mechanism by which carbon dioxide is converted to protoplasm(1)(2) but it is obvious that the carbon dioxide must undergo a certain amount of reduction, thereby becoming a final hydrogen acceptor. The energy for synthesis is derived by the nitrification reaction.

Nitrification in aerobic biological treatment systems occurs primarily at the end of the aeration tank or at the bottom of the trickling filters. For a long time it was thought that this tail end nitrification resulted from the fact that organic matter inhibited nitrification. The concentration of organic matter in biological waste systems is not sufficiently high enough to inhibit the nitrifiers.(5) But the concentration of organic matter is often high enough to cause the heterotrophic bacteria to drop the oxygen level below that required for the nitrifiers. The low dissolved oxygen content prevents nitrification; but stabilization of the organic matter allows the dissolved oxygen level to rise and nitrification to begin. Since nitrification occurs only with an excess oxygen which results only when the organic level is low, it is not surprising that the presence of nitrates was considered a sign of a stable effluent in the early days of sanitary engineering.

Nitrites and nitrates are both readily available hydrogen acceptors once the dissolved oxygen drops below the level for their formation. Nitrates are the most easily reduced and are converted to nitrites. Very little is known about the exact mechanism of the reduction of nitrites other than the fact that the nitrites are reduced with the formation of nitrogen gas rather than ammonia. The over-all reaction of nitrates to nitrogen gas is known as denitrification. Denitrification is probably most important in activated sludge systems.(6) Activated sludge systems treating high nitrogen wastes to a high degree of stabilization produce large quantities of nitrates. The activated sludge is separated from the liquid by sedimentation in a final clarifier. The active microorganisms in the sludge continue to metabolise even though they have left the aeration tank. As the sludge settles, there is a demand on the dissolved oxygen in the settling effluent. If the settled activated sludge is not removed rapidly, the demand for oxygen can exceed that contained in the liquid surrounding the sludge. The nitrates become the next available source of oxygen after the dissolved oxygen has been reduced and the reduction continues until nitrogen gas is evolved. The small gas bubbles become lodged in the sludge and can lift the compact mass of settled sludge off of the floor of the settling tank. Rising sludge prevents adequate sludge being available for return and can cause considerable operating problems. If the evolution of gas is great enough, the sludge will rise to the surface before discharging the gas. The sludge must then settle all over again and part will be lost in the effluent. Rising sludge problems can best be prevented by rapidly removing the activated sludge which settles in the final settling tank. Reducing the amount of nitrification might prevent sufficient gas from forming to cause the sludge to rise. This is done by reducing the excess aeration by cutting the aeration period or by the use of tapered aeration.

Nitrates have long been used as a source of oxygen in lagoons or in streams where wastes create short periods of oxygen deficiencies. It is possible in wastes containing high nitrates to completely stabilize the wastes without the addition of dissolved oxygen. Plain mixing is required to keep the

microorganisms in suspension and in contact with fresh wastes and nitrates. The mixing must be sufficient to dispel the nitrogen gas formed or a rising sludge problem will result.

Sulfates

Sulfates are a common form of chemically combined oxygen and are the source of many problems in waste disposal systems. Sulfates are present in high concentrations in the carriage water in many domestic sewages and in many industrial wastes. Sulfates are reduced only after the dissolved oxygen, nitrates and nitrites have all been reduced. The heterotrophic bacteria which are metabolising the organic matter must have a hydrogen acceptor for $DPNH_2$ as discussed previously and can reduce sulfates all the way to hydrogen sulfide. Excess production of hydrogen sulfide can lead to nuisance conditions and corrosion of equipment.

In domestic sewage systems, sulfate reduction is primarily important in the collection lines and in the anaerobic digesters. The collection of sewage from a large community for discharge to a treatment plant requires a large network of pipes. There is a definite period of retention of the sewage in the collection pipes before reaching the sewage plant. The microorganisms in the sewage are metabolising during this collection period. In most sewers the dissolved oxygen is quickly depleted and the sulfates form the greatest available source of oxygen. If the collection system is long and the time of retention in the pipes is long also, considerable biological action can take place with the production of large quantities of hydrogen sulfide. Agitation of the flowing sewage can cause the release of hydrogen sulfide to the outside environment, resulting in nuisance conditions. Some of the hydrogen sulfide will dissolve into the moisture at the crown of the sewer and be oxidized by autotrophic bacteria to sulfuric acid.(7) The sulfuric acid will attack the sewer pipes unless corrosion resistant materials are used. H_2S problems in sewers can be controlled by reducing microbiological action through chlorination before sulfides have become a problem. Corrosion resistant sewers such as vitrified clay should be used wherever high sulfate waters are encountered. High sulfide sewage will cause a definite oxygen demand in aerobic treatment devices and will retard proper stabilization. (Fig. 3)

Sulfates are an important hydrogen acceptor for the acid-producing heterotrophic bacteria in anaerobic digesters. The oxygen content of the carbohydrates, proteins and fats contained in the digesting sludge is not sufficient to produce complete stabilization. Although the sulfate content of digesting sludge is limited by the quantity in the carriage water, these sulfates can accept 10 hydrogen atoms per mole of sulfate reduced to sulfide. The sulfide collected with the gas can cause corrosion in the gas collection system and in duel fuel engines if it is not removed.

Many industrial processes use large quantities of sulfuric acid which appears in their waste waters as either free acid or as neutralized salts. Treatment of such high sulfate wastes by biological means requires definite care to see that the process used has sufficient oxygen to prevent the reduction of sulfates. Otherwise nuisance odor conditions will result. This is especially true of trickling filters which are actually a combination aerobic-anaerobic device.

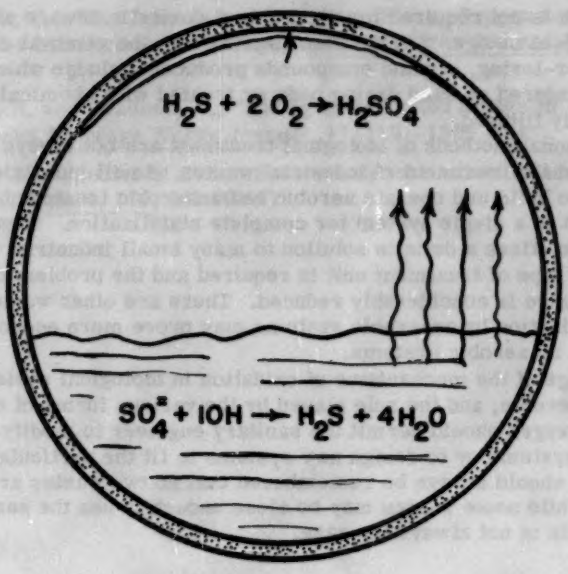

Figure 3 - Sulfur Transformations in Sewers

DISCUSSION

Chemically combined oxygen is essential to biological waste disposal processes; for without it there could be no waste stabilization. The quantity of oxygen contained in the organic matter being stabilized will reduce the amount of additional oxygen required. It is for this reason that high oxygen containing compounds such as carbohydrates or short chain acids are so easily stabilized by both aerobic and anaerobic treatment process and that low oxygen containing compounds such as fats and oils are difficult to stabilize biologically.

Complete stabilization of organic matter by aerobic systems is possibly because of an excess of dissolved oxygen. This can be readily obtained in practice in activated sludge systems but cannot be obtained in trickling filters, as trickling filters are not true aerobic devices but combination aerobic-anaerobic devices. Complete stabilization should not be confused with complete oxidation. Complete oxidation occurs when the organic matter in the wastes has been converted to protoplasm with a definite fraction being oxidized to produce the energy for protoplasm production. Complete oxidation requires the oxidation of the protoplasm formed which is a slow process compared with the oxidation for the formation of protoplasm. Standard practice uses anaerobic digestion for stabilization of the protoplasm produced in aerobic systems. Anaerobic digestion is not a complete stabilization process as there is insufficient oxygen for complete stabilization. But complete

stabilization is not required for digestion of domestic sewage sludge and excess biological sludge. Partial stabilization with the removal of the hydrophilic, water-loving, organic compounds produces a sludge which can be readily dewatered on sand drying beds or treated with chemicals and mechanically filtered.

Conventional methods of biological treatment are not always best suited to industry and the treatment of industrial wastes. Small industries often cannot afford to build and operate aerobic and anaerobic treatment devices and must resort to a single system for complete stabilization. Complete aerobic stabilization offers a definite solution to many small industrial waste systems as only one type of treatment unit is required and the problem of disposal of digested sludge is considerably reduced. There are other wastes where complete stabilization by anaerobic systems may prove more economical and just as efficient as aerobic systems.

Knowledge of the mechanisms of oxidation in biological systems, both aerobic and anaerobic, and the role played by the various forms of chemically combined oxygen should permit the sanitary engineer to modify conventional treatment systems or to design new systems to fit the particular waste being treated. It should always be remembered that no two wastes are exactly alike and while some wastes may be close enough to use the same treatment process, this is not always the case.

SUMMARY

Chemically combined oxygen is of utmost importance in the stabilization of organic wastes by biological systems. The oxygen contained in water is used for the production of carbon dioxide in oxidation reactions and as a supplemental source of oxygen in protoplasm. The oxygen contained in organic compounds is also used for the production of carbon dioxide and for protoplasm. Nitrite, nitrate and sulfate oxygen, on the other hand, act as hydrogen acceptors with the production of water. Aerobic treatment systems primarily use dissolved oxygen as the final hydrogen acceptor but will resort to nitrates, nitrites and sulfates if there is a deficit of dissolved oxygen. The microorganisms in anaerobic systems require chemical oxygen to bring about stabilization of organic matter and use the same type of oxidation system used by the microorganisms in aerobic system.

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DIVISION ACTIVITIES

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

1955-56 DIRECTORY

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OUR PROFESSIONAL PRESTIGE

David H. Howells

Contemporary engineering journals have carried a seemingly endless series of articles dealing with the professional status of engineers. The fact that we have not accomplished full public acceptance of our concept of our rightful place in the sun has been well-established; however, there exists a marked divergence of opinion among engineers as to what our goal should be and how we can best attain it.

Most of us will agree that there is no single cure-all for the ills which plague our collective engineering ego. Our search for a solution to this problem has even prompted some to query members of the legal and medical professions as to the secret of the high degree of professional prestige they enjoy. Yet, we realize that the relationship between engineer and client and the nature of engineering services are so utterly different from those encountered in the practice of law and medicine that we must concentrate our efforts on our own ground if any real progress is to be made.

The engineer, by and large, lacks those feelings of enthusiasm, devotion, and jealous regard for the honor of his profession that must be developed to a high degree if engineering is to be considered a profession, rather than applied technology. He leaves engineering school filled with pride at being able to identify himself as an engineer, but with very little knowledge of the responsibilities a profession requires of its members. His esprit de corps is, too frequently, based upon an immature picture of the engineer as a surveyor-extraordinary, complete with slide-rule dangling at the hip. Add to this, the rather universal concept that professional registration is necessary only for those in private practice, and you have a shallow foundation for professional development.

Those of us who are already members of the profession should impress upon the new engineer that he has a definite responsibility to participate in professional activities, and we should then make a place for him so that he is immediately utilized within the group. He should be actively encouraged to serve his professional society by committee or other task group participation and not be left adrift in a sea of anonymity. A rich source of energy and new ideas lies within the large group of younger members of our Society. Under aggressive and thoughtful committee leadership, these men can be used effectively to pump new and vital blood into the arteries of the Society.

If not utilized by his own profession, the more energetic and resourceful engineer will throw his desire to serve into other channels and may adopt an attitude of indifference toward professional activities outside the immediate requirements of his employment. He will not act like a professional man, simply because he will not feel like one, and the public will accept him accordingly.

Engineering schools, engineer employers, and supervisors should make a concerted effort to do all within their power to develop a professional mental attitude, as well as technical ability, in the young engineer. He should be made to realize that professional bearing and conduct instill confidence in an engineer's competency and are essential to successful engineering practice. The new graduate should be required to attain professional registration at the earliest possible time following graduation. Perhaps, more financial recognition by employers of registration would help to stimulate its attainment.

Above all, we should accept the new graduate as an engineer and not as a sub-professional employee.

Another factor which would have significant, long-term effects upon engineering prestige is that of increased extra-professional activity by engineers in the management of our schools, municipal affairs, and in assuring good government at all levels. If physicians and lawyers have the time for extra-professional activities in local affairs, engineers have little excuse for not making a better showing. Ex-President Herbert Hoover has made tremendous contributions toward increasing the engineer's prestige through his great service to his country and the fact that he has concurrently always proudly worn the mantle of an engineer.

Surely, an all-out effort to improve the professional attitude and esprit de corps of engineers, a stronger program to utilize the energy and talent of younger engineers within Society activities, and the encouragement of public service by engineers would have a profound influence upon the degree of prestige which we enjoy. If we are really concerned about our professional status, we must be willing to put forth the effort required to effect constructive changes wherever possible.

EDITOR'S NOTE: The following editorial from the North Dakota Water and Sewage Works Conference OFFICIAL BULLETIN has been recommended to your editor for reprinting in this section of the Sanitary Engineering Journal. Jerome H. Svore, the editor of the Official Bulletin, has agreed to the reproduction of this editorial which we believe will be of interest to all Sanitary Engineers.

EDITORIAL

The Health Department quite often receives inquiries regarding consulting engineering services for municipal utilities. Many cities feel it is an unnecessary expense and that somehow or other they should be able to advertise for bids to cover a job without going through a consulting engineer.

In requesting an engineer's service, the municipality realizes it must pay a certain percent of the total cost of the job for his professional services. They often feel that there should be a fixed fee, and we do not intend to argue that point. We might mention in passing, however, that an engineer will gladly do a job for a fixed fee, but would certainly base that fee on his estimate of the maximum cost of the job. The city in that case would suffer when the contract was let for less than the estimate.

There are a couple other points, however, that bear mentioning on this matter of engineering services. North Dakota has a law on the books requiring the services of a licensed professional engineer on any job that runs over $2,000. The experience of this office has certainly proven the justification of that law. Tied in with that law is the approval of plans and specifications by the State Health Department. Prior to the time that approval was required, and prior to the licensing of engineers, there were many installations made throughout the State by pseudo engineers that cost municipalities unnecessary thousands. We should realize, too, when we consider an engineer's fee that there are certain types of municipal installations that are highly complicated, technical structures that require a higher fee than is necessary for the ordinary routine type of installation. Might we suggest that you employ an

engineer on the same basis that you secure other types of professional talent, such as medical and legal. If your appendix needs removing, you want a good doctor, not a cheap one. We do not mean to insinuate that a good engineer doesn't make a mistake, but when you are spending thousands of dollars, even though the job may look simple to you, the protection of good engineering services with air-tight plans and specifications can save you many times his fee in securing a facility that will work.

There is little in this world today that is not directly or indirectly produced as the result of the tremendous advances brought about by the engineering profession. Engineers, as a group, are to be commended for their part in the advancement of this technology and for their high integrity, as a group. There are individuals, however, that should come in for a little knuckle rapping for what we feel is a complete lack of ethical procedure. When a municipality employs an engineer, they do so in order that he may protect their interests, not only technically, but financially. We have seen engineers use their stamp on plans and specs. prepared commercially in such a manner that no one but that concern can bid on them. That definitely is not in the best interest of the community. That engineer is working for the commercial concern and not for the people he is supposed to serve. We do not refer here to the perfectly legitimate procedure of an engineer requesting technical assistance from various commercial concerns in determining the efficiency and method of operation of their specific type of equipment. In hiring an engineer, you must, of course, accept his judgment in many instances on which type of equipment he feels is the best, but there is considerable difference in drafting plans around certain equipment or equal, and merely rubber-stamping someone else's work.

This office does not recommend one engineer over another as long as they are legally licensed to do work in North Dakota. All engineering firms should realize, however, that, as representatives of a professional group, the most valuable assets they have are their integrity and ethics. When monetary gain overshadows a sincere desire to minister to people, they are no longer a professional group.

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SANITARY ENGINEERS IN THE HOSPITAL INDUSTRY

Donald L. Snow

Our Nation's sixth largest industry—hospitals—is discovering that sanitary engineers can make real contributions in its operations. Problems ranging from a hospital site location to the actual building design and from plans approval to reviewing a hospital's daily operations, call for solutions from sanitary engineers.

The Federal Hospital Survey and Construction Act, known as the Hill-Burton Act, which provides for Federal hospital construction grants, has given impetus in recent years to a large number of state-operated programs requiring sanitary engineer participation. Chief among these are the hospital plan reviewing and hospital licensing and survey activities within the states. Almost all of these state agencies now directly employ sanitary engineering personnel, especially for the plan review and licensing functions, or look to state health department sanitary engineers for assistance.

To meet minimum standards for hospital construction, yet keep hospital costs down is taking the resources of hospital architects and their engineering associates alike. Sanitary engineers are simultaneously helping interpret these standards and reviewing present equipment and techniques for possible economies. For example, the question of where conditioned air can be recirculated as an economy measure in various areas within hospitals without adverse effects is currently commanding the attention of both sanitary and mechanical engineers. Substitute measures for the chemical disinfection of various hospital wastes represents one traditional technique now being re-evaluated.

New developments in steam, gas, and radiation methods for sterilization, the use of disposable, single-use paper and plastic materials and the introduction of mechanical washing equipment for clinical glassware are some of the newer technologies sanitary engineers are helping bring to hospital operations. Other areas of concern include air sanitation in hospital surgeries, infant nurseries and other critical areas, where engineered combinations of air filtration, ultra-violet equipment and air circulation patterns are required. Zeolite softening plants and, more recently, ion-exchange units, are being increasingly installed in hospitals for laundry water and boiler feed water treatment. Demineralizers for pre-treating water for large stills in hospital pharmacies and sterile preparations facilities are beginning to be employed. In such areas as these, sanitary engineers are bringing to bear their combined biological science and engineering backgrounds.

A still newer activity for sanitary engineers and one which is still hardly explored are the environmental needs for nursing and rest homes. The necessity for a low first cost of construction without diminished standards of sanitation are requiring the use of substitute materials wherever possible.

Another expanding activity for sanitary engineers lies in directing environmental sanitation programs in universities and research laboratory establishments. Most of these have hospitals and other clinical facilities directly connected with them. Today, a number of the Nation's largest universities have environmental sanitation offices with complete complements of personnel trained in such fields as radiological safety, industrial hygiene, housing, food sanitation, and insect and rodent control under the general direction of sanitary engineers.

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DID YOU KNOW THAT—

Russell W. Hart, Sanitary Engineer Director, engineer officer of the Public Health Service Commissioned Corps, who has been Regional Engineer at the Kansas City Regional Office since April 1951 transferred to the San Francisco Regional Office effective June 11, 1956. Mr. Hart obtained his B. S. degree in civil engineering, with major in sanitary engineering, from the University of Michigan in 1938, and joined the Service in May 1941.

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INDIANA AND WISCONSIN WATERWORKS OPERATORS LEARN MORE ABOUT RADIOACTIVITY

Leo Louis, Jr., A. M., ASCE, Vice President and General Manager of the Gary-Hobart, Indiana, Water Corporation, and the Indiana State Board of Health sponsored a two-day course in the elements of radioactivity for Indiana waterworks operators on May 21 and 22, 1956. The course was given at the Indiana University Extension Center in Gary. Instruction was provided by training personnel from the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio.

A similar course was given for Wisconsin waterworks operators at the Milwaukee water treatment plant on May 17 and 18, 1956. Co-sponsors of this course were the Wisconsin State Board of Health and John Keerslake, Water Plant Superintendent, Milwaukee.

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Ed Thoits has succeeded Fred Nelson of the Dorr-Oliver Company in the Northern California area. Frank Hill is a new member of the organization, working with Ed.

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One of the regular members of the Sanitary Engineering Division, San Francisco Section, is a woman engineer. She is June Brevdy of the U. S. Corps of Engineers.

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PURDUE INDUSTRIAL WASTE CONFERENCE

The Eleventh Annual Purdue Industrial Waste Conference held May 15-17, 1956, at Lafayette, Indiana, was attended by 370 representatives of industry, education, and government. Registrants from 31 states and Canada were in attendance. A wide variety of industrial waste subjects was discussed in the 54 papers presented. For the first time, a large portion of the program was devoted to air pollution problems at this conference. Twelve papers dealing with this subject were presented. Conference Chairman was professor Don E. Bloodgood, Member, ASCE.

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A.S.C.E. MEMBER HOST TO WATER PRODUCERS

On May 31, 1956, Walter A. Peirce, Member, A.S.C.E., and Manager, Racine, Wisconsin, Water Department, was host to the annual homecoming meeting of the West Shore Water Producers Association in Racine.

The Association, composed of water plant operators and water utility executives from communities bordering Lake Michigan in Wisconsin, Illinois, and Indiana, is a live-wire organization devoted to the solution of common water supply problems.

At this meeting, representatives of each city were called upon to describe what is being done during the current year to meet higher water demands and to avoid the summer water shortages experienced in 1955.

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Richard H. Bogan, University of Washington, assistant Professor of Sanitary Engineering, will work at Richland this summer in radiological work.

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G. H. Dunstan, Professor of Sanitary Engineering, State College of Washington, will serve on a panel to discuss undergraduate sanitary engineering education at the annual meeting of the American Society for Engineering Education.

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The Pacific Northwest Sewage and Industrial Wastes Association will hold its annual meeting at Boise, Idaho, on October 3-5, 1956. The following members of A.S.C.E. are serving on the Program Committee: H. C. Clare, Public Health Service, chairman; G. H. Dunstan, State College of Washington; M. W. Runyan, Stephans and Thompson Consulting Engineers; and R. O. Sylvester, University of Washington.

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The Pacific Northwest Industrial Wastes Conference was held at the State College of Washington, Pullman, on April 26-27, 1956. For the first time an executive committee was created to take care of interim planning between conferences, which have been held at the University of Washington and Oregon State College, as well as W.S.C. A.S.C.E. members who were selected this year include: E. F. Eldridge, Director, Washington Pollution Control Commission; G. H. Dunstan, State College of Washington; Fred Burgess, Oregon State College; and R. O. Sylvester, University of Washington.

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Roger James, who has been working toward a master's degree in sanitary engineering at the University of California, has returned to his duties as District Engineer, Washington State Department of Health, with headquarters in Spokane.

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Robert C. Merz, Associate Professor of Civil Engineering, University of Southern California, Los Angeles, will present a paper on "Irrigation with Sewage Effluent for Agricultural Purposes" before the American Chemical Society, Division of Agricultural and Food Chemistry, at the Spring Meeting in Dallas.

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Under the chairmanship of R. G. Kincaid, the Program Committee is rapidly formulating the program for the Sanitary Conference to be sponsored by the Kansas City Section of the ASCE this fall. Conference dates are November 19 and 20, to be held at the Continental Hotel in Kansas City. L. W. Bremser is the Board Contact Member.

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You have no right to be disappointed that news from your section of the country does not appear in this section if you fail to submit it to the editor.

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A new brochure on graduate offerings leading to the Ph.D. degree in Sanitary Engineering has just been released by the University of Florida. The undergraduate and master's degree programs are also described in this publication. The fields of sanitary engineering now covered in the graduate program include water supply and wastes engineering; sanitary engineering design; and general sanitary engineering, including environmental sanitation, air pollution, and radiological health. Copies of the brochure may be obtained on request to Prof. John E. Kiker, Jr., Civil Engineering Department.

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Kansas City Section members in attendance at the Knoxville Convention included Ray E. Lawrence, Glenn E. Hands, and Kenneth S. Lane.

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The first joint training courses for Reserve sanitary engineer officers of the commissioned services—Army, Navy, Air Force, and Public Health Service—were given at the PHS Robert A. Taft Sanitary Engineering Center in Cincinnati in June. Developed jointly by the Department of Defense and Department of Health, Education, and Welfare, in collaboration with the National Research Council, these courses were designed to bring Reserve engineer officers normally on inactive status.

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The Sanitary Division of the San Francisco Section, ASCE, recently heard guest speaker Weldon L. Richards, AM, ASCE, member of the newly formed Bay Area Air Pollution Control Board, outline the status and purposes of the Board. This Board is unique in that it is a single special agency having jurisdiction in six counties. The first actions of the Board will be to select a control officer and an executive officer and then to obtain basic information on the smog problem in the Bay Area. This new Board promises to give more opportunities for sanitary engineers in the air-pollution field.

The officers of the Sanitary Division, San Francisco Section, ASCE, for 1956 are: Donald G. Larkin, Supervising Sanitary Engineer, East Bay Municipal Utility District, Oakland, California, Chairman; Jack Pratt, Process Engineer, San Francisco, Vice Chairman; George White, Wallace and Tiernan Company, San Francisco, Secretary.

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Fred Nelson of Dorr-Oliver Co., Oakland, California, has been transferred to the East, and his position has been taken by Ed Thoits.

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Ralph Stone, Research Associate, University of Southern California, Los Angeles, presented a paper on "Application of Waste Water Reclamation to the Mining Industry" at the annual convention of the AIME in New York.

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Prof. Earman Pearson of the University of California spoke on the "Submarine Outfall Disposal of Sewage" at the May meeting of the SED, San Francisco Section. Forty members, one of our largest groups, attended this meeting which was held in San Francisco. This type of sewage disposal has become widely used on the West Coast during the past few years. Prof. Pearson and others have worked on the preliminary plans to adapt the new Hyperion sewage disposal plant in Los Angeles to this system.

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"The Design of Storm Water Inlets" published by Johns Hopkins University is the report of findings derived from the Storm Drainage Research Project conducted at the Johns Hopkins University during the years 1949 through 1955. This work was sponsored and financed jointly by Baltimore City, Baltimore County, and the State Roads Commission, all of Maryland.

"The Design of Storm Water Inlets" is devoted exclusively to the problem of inlet design. Findings are presented based on model tests of many types of inlets. Included in its 193 pages is information on: curb, gutter, deflector, combination, sump, and multiple inlets in depressed and undepressed settings, and on various crown slopes and gutter grades. Also included are: inlet rating curves and tables, nomographs and charts of gutter flow parameters, empirical equations and illustrated examples of their use, findings, and recommendations. The book ($5 per copy) can be obtained by writing to: Paul Bock, Project Engineer, Department of Sanitary Engineering and Water Resources, The Johns Hopkins University, Baltimore 18, Maryland.

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Several of the SED members took part in the convention of the California Sewage and Industrial Wastes Association in Santa Rosa, California, in April. Herb Foster, Supervising Sanitary Engineer of the California State Department of Public Health, Berkeley, and President of the California Sewage and Industrial Wastes Association, presided over the convention. A record number, over 500, attended this convention. A paper on "Some Specific Design Considerations in the Los Angeles Sewerage Program" was presented by Richard Pomery, Consulting Engineer, Pasadena, California, and Richard Kennedy, San Francisco.

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CAREER GUIDANCE FOR FUTURE ENGINEERS

The engineering profession took a leading part in the first day's activities of the Rich Township High School's new Annual Career Week Program in Park Forest, Illinois, on April 23, 1956. Approximately 200 senior students attended the engineering panel presentation of career opportunities in the various fields of the engineering profession. Representatives of Civil, Architectural, Mechanical, Electrical, Chemical, Aeronautical, and Metallurgical Engineering took part in the program. David H. Howells, AM, ASCE, Chicago, represented Civil Engineering on the panel. He reports that student interest was high and that the types of questions asked were as comprehensive as could be expected from only a five-minute discussion of each engineering

specialty by the panel members. The Career Week Program was judged a successful undertaking and will be repeated on an annual basis in the future.

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EDITOR'S NOTE: Here is an idea that each and every member can use to make a contribution. Why don't you contact the principal of your high school and offer your services NOW.

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A. H. Wymore, who is Chairman of the Water Pollution Control Committee, Missouri Water and Sewage Conference, presented a paper on pollution control legislation at the June 12 meeting of the Missouri Public Health Association in Kansas City. At the dinner following this meeting, L. E. Ordelhiede presented the W. Scott Johnson Award to Dr. John Williams of Springfield, Missouri.

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Paul E. Haney, Melvin P. Hatcher, and Russell P. Culp served on a panel at the annual AWWA Convention in St. Louis, discussing before the Management Section the submission of plans and specifications to Control Authorities.

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Wallace W. Bergerson resigned as Associate Engineer for Sanitary Engineering work in the Seattle City Engineering Department, and is now a consultant on lighting with Verd-a-Ray Corporation.

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A state-wide conference on water quality was held in San Francisco May 24. The conference, sponsored by both official and nonofficial agencies, concentrated on the problems of physical and chemical quality of water necessary for various uses and the legal problems associated with water quality. Governor Knight addressed the luncheon session on the subject "Water and Twenty-five Million People."

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The Sanitary Engineering Committee of the Seattle Section, ASCE, under chairmanship of John H. Bright, has a very active program. A recent meeting on sewage disposal problems in the Seattle metropolitan area drew an attendance of 70 persons. They are also working with the city on implementation of the recently approved program for major improvements in the city's sewer system; and on development of a state water resources policy.

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It is planned that Dr. E. R. Hendrickson, Associate Professor of Civil Engineering, University of Florida, will be assigned by the Public Health Service to work with Emil C. Jensen, Chief Sanitary Engineer, Washington State Department of Health, for the summer, on an evaluation of the air pollution problem in the State of Washington. This will be a cooperative Federal-state study similar to one underway in Connecticut.

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Alex Diachishin, principal engineer for the Interstate Sanitation Commission, and secretary of the ASCE New York Metropolitan Section, Sanitary Engineering Division, is resigning his present position to enter the consulting engineering field in the New York area.

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The bill to extend and strengthen the Federal Water Pollution Control Act passed the House of Representatives by a vote of 339 to 31 on June 12. At the time material was prepared for this journal, the bill was scheduled to be considered by a Conference Committee since the House of Representatives amended the bill originally passed by the Senate.

The Act to be considered by the Conference Committee contains provisions for an increased research program, collection of basic data, grants to State water pollution control agencies for program expansion, grants to municipalities for construction of facilities, and provision for Federal participation in enforcement activities on interstate streams at the request of a downstream State.

The most controversial provision of the Act pertained to the provision of grants to municipalities for construction. This provision authorizes $50 billion per year for 10 years to assist communities with the construction cost of treatment facilities. Assistance is limited to one-third of the cost of the facilities or $300,000 whichever is smaller. Fifty percent of the amount appropriated must be used for grants for the construction of treatment works servicing municipalities serving 125,000 population or under. For those unfamiliar with the Federal procedure it should be pointed out that the Act under consideration is an authorization only and that funds are not available unless appropriated annually.

The necessity for preparing copy for this Journal in advance will result in final action being taken on the Act by the time this is printed. The next issue will, therefore, carry information on the final action.

LETTERS TO THE EDITOR

EDITOR'S NOTE: The April issue of this journal carried a letter to the Editor on the problem of waste disposal which was included since it was believed it would stimulate a smile or two. Your Editor has debated the propriety of including the present letter but has finally decided to include it and hopes that the membership will accept the letter for the humor it contains. However, it should be noted that no additional contributions are solicited on this subject.

Dear Editor:

The Letter to the Editor in the SED Newsletter, April 1956, presenting the problem of body waste disposal in space travel, fascinated me. The writer deserves commendation for his foresight (typical of sanitary engineers) in giving thought to the solution of a problem before it arises. While I do not qualify as a science fiction writer, he invited suggestions and, after thinking it over, I have one.

The writer is pessimistic on one point—that shoes are about all we can magnetize. That is not so. They say this is an electronic age so no opportunities to exploit the science to our advantage should be overlooked. I

suggest that each space ship carry along one or more highly magnetized steel balls (size to be determined after due consideration of all pertinent factors) which can be jettisoned in space one at a time when needed. Then include a generous component of iron filings in the diet of the space travellers (after all, they are going to have to be tough people, aren't they?) which, of course, will pass out with the excrements. These will be discharged from the hull of the ship and, instead of floating around in limbo forever as the writer contemplates with esthetic misgivings, they will be immediately attracted to and retained by the magnetized steel ball. The ball will thus become a sort of "interplanetary space privy."

As for final sanitary disposal, three major forces operate in our behalf. First, the matter of sheer space alone will cause many light-years of dispersion between "privies." Second, it's cold up there and the intestinal pathogens in the feces will die off rapidly. Third, the bombardment of the pathogens by cosmic rays should do something.

It is clear that this proposal complies with the basic principle of good environmental sanitation which says that it is better to concentrate wastes in one pile for ultimate disposal by natural forces than to scatter them around the landscape.

As another strong proponent of cleaner space travel, I remain

Respectfully yours,

Dear Sir:

The movies may have cinemascope—but the TV has stethoscope!

There is much more than just humor in this comment. The medical profession has utilized this media to advertise itself in a very advantageous and dignified manner. And the legal profession is not far behind. But where, oh where can the engineers be???

The engineering profession could stand to take a few notes from the TV screen and see how it, too, could enhance its position as a profession.

I was very happy when I finally saw an announcement in the local papers about a TV show dealing with Air Pollution in New York City. I watched for it with eager anticipation. The program turned out to be a panel discussion which was fine for a technical group or a group of specially interested citizens.

Dr. Leonard Greenburg, Director, Division of Air Pollution, New York City, was very lucid in his presentation of the problems of air pollution in New York City. William T. Ingram, member of Engineering Research Division, NYU, gave an excellent review of some of the aspects of air pollution research being carried on under his auspices. The third and fourth members on the panel were from the hard coal institute and from real estate interests in New York.

The two technical men on the panel outdid themselves. They were confident, well prepared, clear and concise. They did a thorough, competent, technical job. But how many people of the TV audience are out to watch technical material?

It is time more conscious effort were expended to put glamour into Civil Engineering, and particularly in our own Division. Recently the Water and Sewage Works Manufacturers Association hired a firm of public relations experts to educate the Nation as to the value of good and sufficient water, in terms of higher rates. Although our reasons are not directly monetary, there is so much that can be done to bring the sanitary engineering profession to the

fore. The emphasis should be that the sanitary engineer is a valuable and necessary member in the development and upkeep of safe, healthy communities.

Ever since the Donora, Pennsylvania, episode in 1948 there have been increasing efforts on the part of communities to enforce more stringent regulations dealing with air pollutants.

Usually the visible smoke and fly ash are the first areas which receive concentrated attention. Invariably this has led to regulations pertaining to incinerators.

In order to develop a more complete basis of fundamental knowledge as well as more effective designs, the Incinerator Institute of America is initiating an incinerator research program with New York University. Effective June 1, 1956, the I.I.A. began a research program on an impartial industry basis, making available to all participating manufacturers information on fundamentals as well as equipment for proper and effective incineration.

Sincerely,

Please cooperate by sending news items of interest to:

Paul W. Reed, EDITOR
Division of Sanitary Engineering Services
Public Health Service
Department of Health, Education and Welfare
Washington 25, D. C.

ASSISTANT EDITORS:

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in issue 1 of the Journal of the Soil Mechanics and Foundations Division.

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AUGUST: 761(BD), 762(ST), 763(ST), 764(ST), 765(ST)[c], 766(CP), 767(CP), 768(CP), 769(CP), 770(CP), 771(EM), 772(EM), 773(SA), 774(EM), 775(EM), 776(EM)[c], 777(AT), 778(AT), 779(SA), 780(SA), 781(SA), 782(SA)[c], 783(HW), 784(HW), 785(CP), 786(ST).

SEPTEMBER: 787(PO), 788(IR), 789(HY), 790(HY), 791(HY), 792(HY), 793(HY), 794(HY)[c], 795(EM), 796(EM), 797(EM), 798(EM), 799(EM)[c], 800(WW), 801(WW), 802(WW), 803(WW), 804(WW), 805(WW), 806(HY), 807(PO)[c], 808(IR)[c].

OCTOBER: 809 (ST), 810 (HW)[c], 811 (ST), 812 (ST)[c], 813 (ST)[c], 814(EM), 815(EM), 816(EM), 817(EM), 818(EM), 819(EM)[c], 820(SA), 821(SA), 822(SA)[c], 823(HW), 324(HW).

NOVEMBER: 825(ST), 826(HY), 827(ST), 828(ST), 829(ST), 830(ST), 831(ST)[c], 832(CP), 833(CP), 834(CP), 835(CP)[c], 836(HY), 837(HY), 838(HY), 839(HY), 840(HY), 841(HY)[c].

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(BD), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1) 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877 (HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1). 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1) 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904 (PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913 (WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941 (SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958 (SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969 (WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

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JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026 (SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

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c. Discussion of several papers, grouped by Divisions.

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